NOV 25 1935

Engineering News-Record

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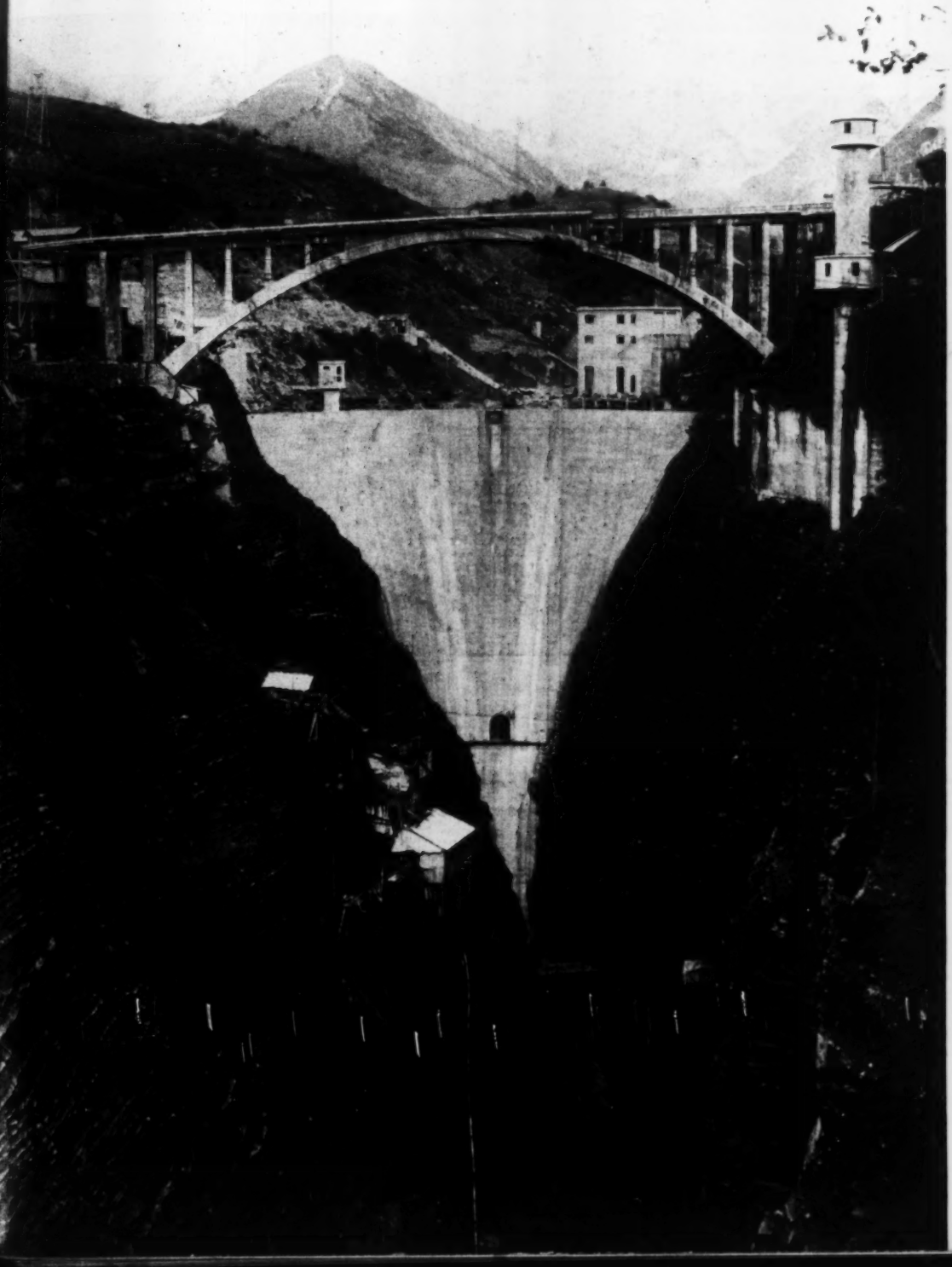

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Sautet Dam in France rises 414 ft. above the floor of a narrow limestone canyon.

. . NOW IT'S ALL WROUGHT IRON

● No pipe as yet has handled calcium chloride brine as well and as economically as wrought iron. The illustration shows another instance where wrought iron in the Chicago Stadium has been put on the job because another pipe material started serious failures in 5 short years. Wrought iron's long record in old ice rinks and refrigeration systems makes it unquestionably the only material to use.

A wealth of technical knowledge, service records and engineering data has been incorporated in the special report on "The Use of Wrought Iron in Refrigeration Systems." This report is available through any Byers Engineer or our Engineering Service Department. It covers not only ice rinks but all kinds of refrigeration systems. A. M. Byers Company, Established 1864. Pittsburgh, Boston, New York, Philadelphia, Washington, Chicago, St. Louis, Houston.

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PROGRESS—*A Ruthless Driver*

JUST about now the American people are looking over the new motor cars. Throughout the land, the manufacturers are displaying the fruits of their genius and skill in research, design and production. Their products reflect the higher values that have been made possible through the application of new materials, refined methods and modern equipment by an industry that never sleeps. Motor user and motor manufacturer alike are highly conscious of technical progress in the science and art of motor transportation.

But it is not the automotive engineer alone who must keep abreast of what is new in materials, methods and equipment. In every department of engineering science progress marches incessantly. Day by day the newspapers chronicle in popular terms the scientific achievement that is recorded more austerely in the proceedings of the technical societies: today a new telephone set, tomorrow new methods of producing gasoline, next day a new plastic of wide utility. To those in industry who are responsible for results, each of these and scores of others sooner or later will mean new opportunities if they be used wisely and aggressively, or new grief if they be permitted to find more effective use at the hands of competitors. The price of industrial success and even of survival is an eternal vigilance by which the engineer insures his up-to-the-minute mastery of the tools and materials of his craft.

MANY of the new materials and methods touch closely the work of the architect, civil engineer and all the others who are responsible for designing and building the structures of today and tomorrow. New treatments, new finishes, new performance, new efficiencies—all these spring from the intelligent and resourceful use of new tools and media. The designer who, during the last few years, may have lost track of all this will be at heavy disadvantage in the days just ahead of us, however soundly grounded in the fundamentals or however experienced in the technique of another day. Technical progress is a ruthless driver; his lash lies heavy over the backs of his laggard disciples.

One function of this and kindred journals is to help the technician keep up with what is new in his field. Through their reading pages they seek to portray the march of principles and practice; through their advertising pages they display the materials, the tools and the services by which these may be applied.

Never has this latter function been more significant than today. With increased need for variety, novelty, utility and efficiency in design and construction in order to justify the investment of new capital and with a sharpened competition for what business there is, wise producers are making available to the engineer and constructor the finest products of which their laboratories, designing rooms and shops are capable. Few are relying simply on the products that satisfied five years ago; few have not so improved the old or developed the new, but that they have for the user of their products a story of technical interest and value to him.

SO the wise producer, offering through his advertising the product of *his own* competitive effort would help his prospective customers with *their* competitive problems; thus he seeks to earn a larger participation in the industrial revival on which we now are entering.

It is just this ruthless march of engineering science, materials and equipment that has made possible our industrial progress to date and that will continue it through the years to come. No engineer or constructor who would hold a place in the ranks can afford, therefore, to ignore the help that is made available to him through the following pages.

Willard Chevalier

ENGINEERING NEWS-RECORD

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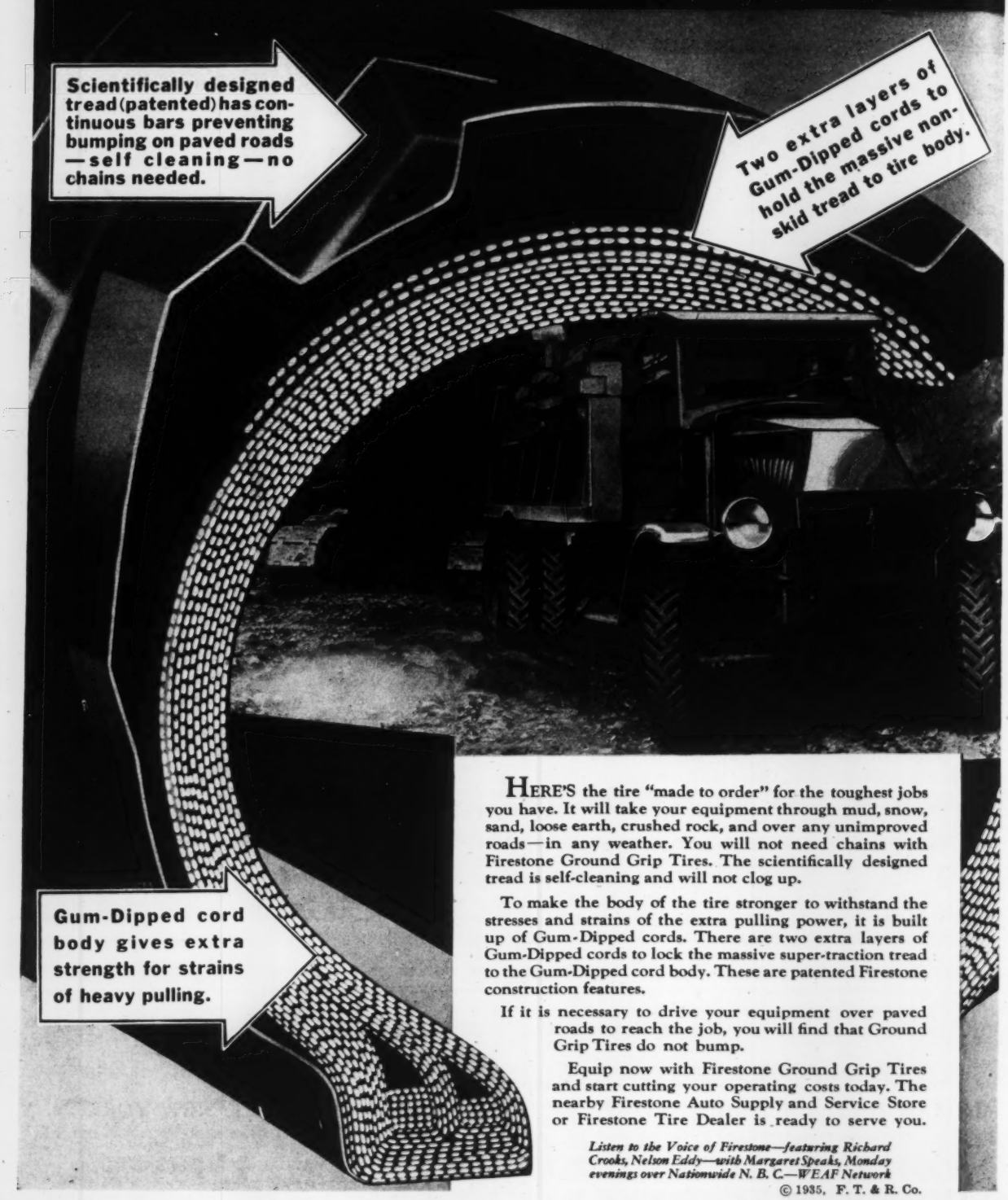
CUT YOUR OPERATING COSTS WITH THE GREATEST TRACTION TIRE EVER BUILT


Scientifically designed tread (patented) has continuous bars preventing bumping on paved roads—self cleaning—no chains needed.
Two extra layers of Gum-Dipped cords to hold the massive non-skid tread to tire body.
Gum-Dipped cord body gives extra strength for strains of heavy pulling.

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If it is necessary to drive your equipment over paved roads to reach the job, you will find that Ground Grip Tires do not bump.

Equip now with Firestone Ground Grip Tires and start cutting your operating costs today. The nearby Firestone Auto Supply and Service Store or Firestone Tire Dealer is ready to serve you.

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In the News:

THE NORTH ATLANTIC SEABOARD suffered extensive damage on Nov. 17 from a northeast storm that coincided with high tides. Coast highways and railway lines suffered heavily as did bulkheads, jetties and boardwalks. One beach sewage-disposal plant is reported to have been unroofed and disabled by being filled with sand.

PRESIDENT ROOSEVELT has agreed to modify the $15,000,000 allotted to the Central Valley project in California. The revision includes $5,000,000 toward the cost of relocating the Southern Pacific Railway in the Kennett Reservoir area.

A SHORTAGE OF ARCHITECTS is reported by the Architects' Emergency Committee of New York, a committee organized during the depression to aid unemployed architects and draftsmen. No such shortage has been found by the Professional Engineers' Committee on Unemployment at New York, but reports from Chicago indicate a shortage of better-trained men in the Middle West.

EXHIBIT SPACE for the Road Show at Cleveland, Jan. 20-24, has been oversubscribed. This is looked on as a most encouraging sign, evidence of renewed confidence on the part of manufacturers serving the highway field.

BIDS on the New Jersey land section of the Midtown Hudson Tunnel, New York, and on the New Jersey plaza have been rejected by the Port Authority as too high. New bids are to be called again in a few weeks.

COMPLETION of the Apex Building, the final structure required to complete the monumental group of buildings in the Triangle at Washington, D. C., is provided for by an allotment of $3,665,000 to the Treasury Department for construction of the new building.

In This Issue:

GROUTING AT NORRIS DAM, to insure watertightness of the underlying dolomite, was carried out according to one of the most elaborate plans ever laid down for such an operation. Results obtained were carefully checked, including the use of a periscope lowered into 5½-in.-diameter holes and sinking 36-in.-diameter holes, which permitted direct examination of subsurface conditions. Possible leakage through the rimrock of the reservoir was studied as part of the undertaking.

BRIDGE-DESIGN experience in the Kansas Highway Commisson office has shown that economies in materials resulting from designing girders and trusses as continuous structures have far exceeded the added cost of making such designs as compared with simple structures.

WIND-DESIGN REPRINTS

The eight articles by W. Watters Pagon on "Aerodynamics and the Civil Engineer" are assembled in a reprint booklet. New knowledge of wind forces, developed by aeronautical engineers, is here first made available to the designers of buildings, bridges and other structures.

Price, 50 cents.

SAUTET DAM IN FRANCE is the highest in Europe and is second only to Boulder Dam among the high dams of the world. The dam is especially notable for its form, for the fact that it is built in a limestone canyon, and for the novel arrangement of its power house.

THE SPECIFICATION of two types of floor construction for the same building gave an opportunity to compare the cost of the more recent junior beam corrugated-iron type with an older form. Results are given in tabular form.

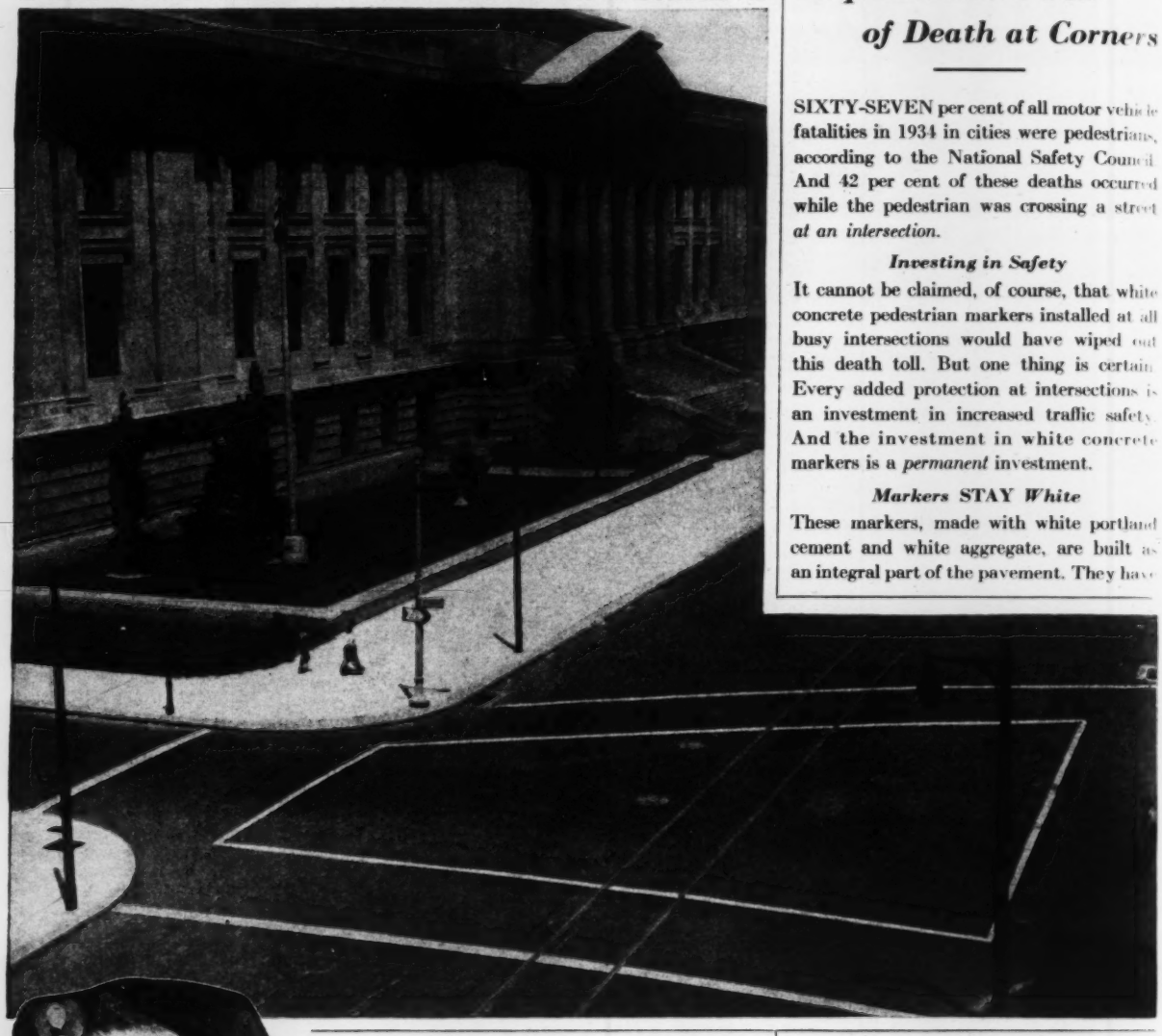
A NEW VISCOSIMETER has been developed that is expected to eliminate all the confusion that has existed heretofore because of the necessity for using different types of instruments for materials of radically different viscosity. This new instrument has been used for all grades of tar-road materials. It is expected to be equally satisfactory for road oils, road asphalts, etc.

AMERICAN ENGINEERS recently completed a new group of deep-water port facilities at Cartagena, Colombia, a port that is rapidly assuming importance in South American trade. Because of peculiar atmospheric conditions and active attack by teredos and termites unusual care was taken in the design of the structures to insure long life.

BECAUSE few roller-gate dams have been built in this country, little information concerning them is to be found in American technical literature. Most of what has been published on this subject is found only in German. In this issue a summary is given of the design analysis used by the War Department in its several navigation dams. The data here given supplements information on the same subject published last year.

WHITE MARKERS GUARD CROSSINGS

PROTECTION FOR PEDESTRIANS



Help Cut Down Toll of Death at Corners

SIXTY-SEVEN per cent of all motor vehicle fatalities in 1934 in cities were pedestrians, according to the National Safety Council. And 42 per cent of these deaths occurred while the pedestrian was crossing a street *at an intersection.*

Investing in Safety

It cannot be claimed, of course, that white concrete pedestrian markers installed at all busy intersections would have wiped out this death toll. But one thing is certain. Every added protection at intersections is an investment in increased traffic safety. And the investment in white concrete markers is a *permanent* investment.

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Valuable Information

For complete information on these indelible white traffic markers that never need replacement, write to Universal Atlas Cement Co. (United States Steel Corporation Subsidiary), 208 South LaSalle Street, Chicago.

Atlas White concrete pedestrian lane markers guard this intersection on 20th Street in Philadelphia by the Benjamin Franklin Memorial. Installation by Eastern Asphalt Co., Philadelphia, under the direction of Dudley T. Cornung, Chief Engineer, Philadelphia.

★ ★ ★

White concrete pedestrian markers can help to cut down intersection fatalities—they are a permanent investment in traffic safety

190

Build Safety into Streets and Highways with

ATLAS WHITE TRAFFIC MARKERS

Made with Atlas White Portland Cement—Plain and Waterproofed

ENGINEERING NEWS-RECORD

November 21, 1935

Thousands of Holes Grouted Under Norris Dam

Holes 10 ft. apart, 30, 40 and 100 ft. deep, and up to 5½ in. in diameter, search out for grout injection every cavity and porous spot in the line rock under body and apron of the great TVA structure

FIG. 1—PERISCOPE INSPECTION of rock conditions through 5½-in. core-drill holes disclosed the need for grouting and the sufficiency of the grouting of the smaller holes.

TO MAKE TIGHT the foundations of Norris Dam and the rim-rock of the reservoir formed by the dam calls for careful studies of leakage and elaborate grouting operations to close leaks. The most extensive of these operations was at the dam proper, to make solid and impermeable the foundation rock under the great concrete barrier. Here a plan of rock solidification by cement grouting was devised and carried out that ranks among the most elaborate undertakings of its kind. The grouting plan and procedure are described here with some account of the reservoir-leakage studies.

As is generally known, Norris Dam is a project of the Tennessee Valley Authority and is located on the Clinch River, one of the principal tributaries of the Tennessee, at a point about 30 miles above the confluence of the two streams. The site was originally selected by the Corps of Engineers, U.S.A. after extensive investigations of numerous locations. This and all other possible sites on the Clinch presented problems in dam foundations and in making reasonably tight the reservoir area developed.

Limestones and dolomites underlie the entire valley of the Clinch and its principal tributary, the Powell. All sites would require a dam and its abutments to rest on these types of rock. Wherever these formations occur in the southeastern part of the United States they have been exposed to millions of years of weathering, and underground solution has produced caves or porous beds. Without exception, these porous beds stop not far below the present river levels, but above them careful provision must be made to avoid weak points in the dam abutments or leakage from the reservoir. The grouting of the dam foundation was conducted so thoroughly that no difficulties are considered possible from the base of this 285-ft. overflow concrete structure. Explorations of the reservoir area have determined that it will be entirely practicable to reduce to close limits leakage in the 83,000 acres that are submerged.

Expert studies agree

The Corps of Engineers conducted extensive investigations of all practical dam sites on the Clinch, with the conclusion that the site chosen was the most desirable. This site was core-drilled by the army engineers, and, on the findings of this drilling, they recommended to Congress that "The dam site and reservoir (Norris) have been found to be satisfactory for the reservoir project proposed."

When the development of the Tennessee River was placed in the hands of the TVA, with specific instructions to build Norris Dam at the site selected by the Corps of Engineers, the authority immediately undertook further much more detailed studies of the foundation of the dam and of the possibilities of leakage from the reservoir area than it had been possible for the Corps of Engineers to make with the funds and time available to that organization.

Dr. Charles P. Berkey, professor of geology at Columbia University, Dr. Arthur Keith, in charge of areal geology for the United States Geological Survey, and Dr. A. C. Swinnerton, professor of geology at Antioch College, were retained to make the desired geological surveys. Their work was also supplemented by the TVA geology staff headed by Major E. C. Eckel. The findings of these geologists are summarized in a preliminary report by Dr. Berkey, of which the following abstract gives the general conditions presented:

The Knox dolomite at the site of the dam is a comparatively hard and substantial rock, varying somewhat in composition and minor physical features from one stratum to another, but it is comparatively massive, compact and sound. Some jointing has occurred along which underground waters have developed occasional seams. These seams are of importance in regard to possible leakage

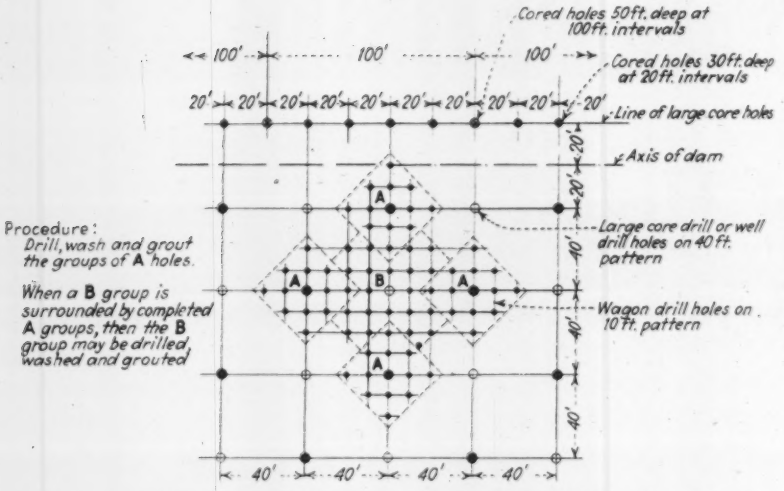

Cored holes 50 ft. deep at 100 ft. intervals
Cored holes 30 ft. deep at 20 ft. intervals
Line of large core holes
℄ Axis of dam
Large core drill or well drill holes on 40 ft. pattern
Wagon drill holes on 10 ft. pattern
Procedure: Drill, wash and grout the groups of A holes.
When a B group is surrounded by completed A groups, then the B group may be drilled, washed and grouted
100'
20'
40'
A
B

FIG. 2—SCHEMATIC PLAN of grout holes. The 10-ft. pattern shown in the A and B groups covered the entire area of the dam and its apron.

rather than concerning soundness and adequacy of foundation. Only very small seams were discovered in the foundation at the dam site. In the abutments several small caverns, as well as some seams, were readily sealed. The dolomitic formation at the dam site is somewhat porous, but the pores are not well connected.

From the determination of these characteristics Dr. Berkey reached the conclusion that there is no doubt whatever of the general competence of the foundation rock. "It is sound enough and substantial enough to support any kind of dam that may be designed for this site, and it is sufficiently resistant to destructive attack to insure the required permanence."

Geologists' conclusions approved

Cooperating with the TVA consulting geologists, a board of consulting engineers was employed by TVA to study rock formations at the dam site and in the reservoir area. This board consisted of L. F. Harza, C. P. Berkey, J. L. Savage, F. W. Scheidenhelm, C. H. Paul and S. M. Woodward. He submitted a report substantially the same as the geologists as to the adequacy of the dam foundation and the practicability of making the reservoir area tight by grouting. Subsequent complete exploration of the foundation of the dam and conditions determined during construction confirmed these conclusions.

Methods of grouting

Explorations having determined that the dolomite foundation of the dam was entirely satisfactory, as previously summarized, the decision was made to clean out and grout all seams so thoroughly that any chance for passage of water under the dam would be eliminated. This work was simplified by the discovery by core-drilling that the rock was entirely free of seams after a depth of about 27 ft. was reached below the bottom of the excavation. At that depth and at various elevations above it, seams filled with clay occurred. None of these seams were continuous. In some cases small closed caverns were found. It was determined that all such seams and openings might be sealed with grout.

The base of the dam, including the apron, extends up and downstream a width of about 400 ft. On a line 20 ft. upstream and parallel with the axis of the dam a line of 5½-in. core-drill holes was put down to secure a grout curtain or cutoff. At intervals of 20 ft. across the foundation these holes were put down to a depth of 30 ft. Every fifth hole in this row, or those on 100-ft. spacings, was drilled 50 ft. deep. Extending downstream to the limits of the apron from this upstream cutoff wall of grout a definite pattern of holes was drilled to obtain complete coverage of the foundation.

Spacing of holes

Spacing of the drill holes through which grouting was done is shown in Fig. 2. First, 1½-in. holes were sunk to a depth of 30 ft. on 10-ft. centers in the squares marked *A*. These holes were cleaned and grouted as fast as sunk. As soon as an area had been thus drilled and grouted, overlapping square layouts of *B* holes of the same size were drilled to a depth of 40 ft. These holes also were cleaned and grouted as fast as a section was ready. These spacings resulted in the entire area being drilled and grouted on 10-ft. centers. As a final check, 5½-in. core-drill holes were put down on 40-ft. centers in both directions to a depth of 40 ft. These larger holes permitted inspection, by a special periscope device developed on the job, of the results obtained in grouting from the 1½-in. holes.

When a group of holes had been drilled, water was forced into them in series at not over 30-lb. pressure. In this manner all clay in the seams crossed by the hole was washed out. Seals on the holes controlled by valves permitted the water to be forced first into one and then another of the holes, so it was possible to reverse the flow and wash the underground seams under pressure. In this manner eventually all the clay was removed, as shown by clear water issuing from the open hole.

Standard grouting equipment (Fig. 4) and methods were used in filling the cleaned holes and seams. Pressure was kept at 30 lb. or less, and grout was pumped in as long as the hole would take it. The amount of grout per hole ranged all the way from 1 to around 200 sacks of cement.

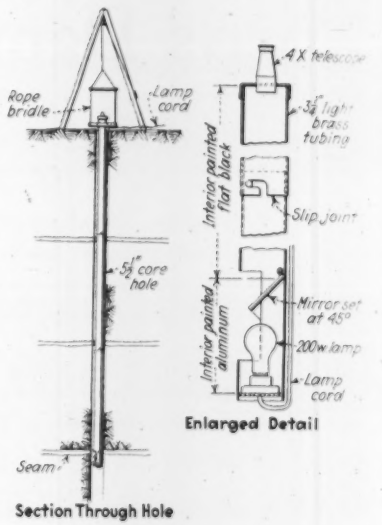


FIG. 3—PERISCOPE used to examine rock conditions in large core-drill holes.

Details of the periscope used in inspecting the results of grouting through the 5½-in. holes are shown in Fig. 3. Sections of 3½-in. brass tubing were made up in 10-ft.-lengths, easily assembled up to 40 ft. A cap on the end of the top section carried a 4-power telescope. In the lower end of the bottom section a mirror was soldered at an angle of 45 deg. Below it in a removable base was a 200-watt electric lamp. At the level of the mirror a section of the tubing was cut away, to expose the face of the drill hole to the mirror. Below the mirror the inside of the tubing was coated with aluminum paint. Black paint above the mirror eliminated reflection.

Suspended from a portable A-frame, the periscope could be revolved at any depth, to permit close inspection from the surface of the face of the rock at any point in a hole. The lighting arrangement gave just as good illumination at the point of inspection as bright daylight.

In addition to the regularly spaced 5½-in core-drill holes for inspection of grouting results, 36-in. core-drill holes were put down at various places over the entire foundation area. Cores from these holes enabled large-scale deter-

FIG. 4—PUMPS INJECTING GROUT into a group of drilled holes. At the left is a pair of hole-drilling rigs.

minations of the results obtained in grouting the foundation. The holes made by them were of ample size to permit a man to be lowered to the bottom of the hole on a boatswain's chair for close study of the face of the rock and grouting as exposed.

This large core-drilling stood out as one of the important features of checking the vast amount of grouting that was done. By means of it and of the inspections by periscope through the 5½-in. holes, it was possible to determine that the grouting had completely sealed the entire foundation. The resulting effect is the same as a solid ledge of rock entirely free from seams and pockets.

Explorations of the abutments of the dam uncovered weak seams and pockets that called for grouting to make them safely tight. As found, these seams and pockets were filled more or less solidly with clay. Few of them showed any signs of having carried water in modern times. All were cleaned out to their full depth, however, and then filled with concrete.

Leakage from reservoir area

Taken as a whole, the risk of leakage from the reservoir developed by Norris Dam is limited to comparatively small areas lying near or along the rim in the lower reaches of the reservoir. The limestones and dolomites are in most places covered heavily with residual soil, usually a clayey blanket that provides a reasonably good protection over the rock surface. Where outcrops of rock occur, or where the blanket is thin or more porous, the waters of the reservoir will reach the rock formation. Some of the latter over the ages have developed water channels, but protection against extension of such channels is considered practicable.

Extensive studies of known water channels in some of the limestones of the reservoir area have been conducted. All important springs below the spillway level of the dam that might be affected by storage of water behind the latter are being checked to determine their behavior under conditions existing before the water rises in the pool. Weirs have been built on the outlets from these springs; recording devices have been placed on those that carry the largest flows or that appear to be supplied by channels extending into the reservoir area.

Records already obtained before the storage of water behind the dam had become high enough to have any effect show that some of these springs are fed from the deep layer of soil over the limestones and dolomites. Others may have inlets some distance from their appearance. A few evidently depend directly on surface outlets through which rainwater drains to a lower level. There is no generality that covers their occurrence or performance.

Possible leakage localized

From the known geology of the reservoir area it has been possible to localize within comparatively limited zones the probability of leakage from the vast pool behind the dam. Out of a total shoreline of some 800 miles the indications are that not more than two miles of exposure present danger of important leakage.

Investigations in the way of surface examination and core-drilling in the zones known to be more or less porous have determined that it is impracticable in advance of the rise of water behind the dam to know just where and how much leakage may occur from the reservoir area through watercourses in the underlying rock. Grouting of all these water courses would entail an immense amount of expense. There is evidence that only a small percentage of them will divert water from the reservoir. Many have been determined to have outlets within the area lower down. Others showing as openings or caverns on the surface have blind ends. On the other hand, there are doubtless important courses that do not show up on the surface but from which water pressure may dislodge the overlying soil.

No advance closures made

Due to these uncertainties, the authority has adopted the policy of undertaking no closure of watercourses in the limestones underlying known weak zones within the reservoir area until the water begins to rise behind the dam. Meanwhile arrangements are being made for very close checks along the entire shoreline of the pool. Observations also will be made of springs and watercourses below the water level both behind and below the dam, to determine any increases in flow other than from rainfall or other normal conditions.

In this manner it is expected to be possible to locate all important leaks from the reservoir as the water level of the pool rises. The strike of the rock formations within the entire reservoir area having been determined closely, it is planned to locate the inlets of watercourses leading from the reservoir that produce evident flows outside. Grouting of the limestone formations in the zones affected is expected to stop most of such leakage. In some cases blankets of clay may be more practicable.

To overcome in this manner the leaks that are expected from the reservoir is expected to require some months, and it may take a year or two to locate and stop some of the leakage. A few minor leaks may become permanent, due to difficulties of determining their source within the reservoir. On the whole, however, ability to make the reservoir efficiently tight is not questioned by the engineering staff of the TVA and its advisors on engineering and geology. Moreover, the authority is confident that the policy of stopping the leaks as they appear will save a large amount of money over a plan of attempting to seal in advance all known weak zones in the basin and the rim of the reservoir.

Direction

A. E. Morgan is chairman and chief engineer of the Tennessee Valley Authority. Carl A. Beck is assistant chief engineer, Barton M. Jones construction engineer, Ross White general superintendent of construction, C. H. Locher construction consultant, and A. J. Ackerman construction plant engineer. F. C. Schlemmer is construction superintendent, and J. S. Lewis was assistant engineer in charge of the grouting operations.

Continuous Spans Favored for Kansas Highways

For three years state highway department has spent half of its bridge budget for continuous structures of rolled-beam, plate-girder and truss type, with savings estimated from 10 to 30 per cent over simple spans

By Geo. W. Lamb
Senior Designer, Kansas Highway Commission, Topeka, Kan.

ON THE STATE highway system of Kansas during the past few years there have been constructed a relatively large number of continuous steel-beam and deck plate-girder bridges and several continuous low-truss bridges. Of the total of 420 bridges costing $4,130,000 built since 1933, over 50 are continuous steel structures with an aggregate cost of over $2,000,000, or about 48½ per cent of the total. Savings over simple span structures are estimated to range from 10 to 30 per cent, or several hundred thousand dollars, so that the use of continuous-type structures has covered increased engineering cost many times over.

Three types of construction that have been found to be economical within the following approximate limits of central span lengths, and dependent somewhat upon the width of the roadway, are as follows:

Rolled beams	Up to 105 ft.
Plate girders	105 to 150 ft.
Low Warren trusses	150 to 200 ft.

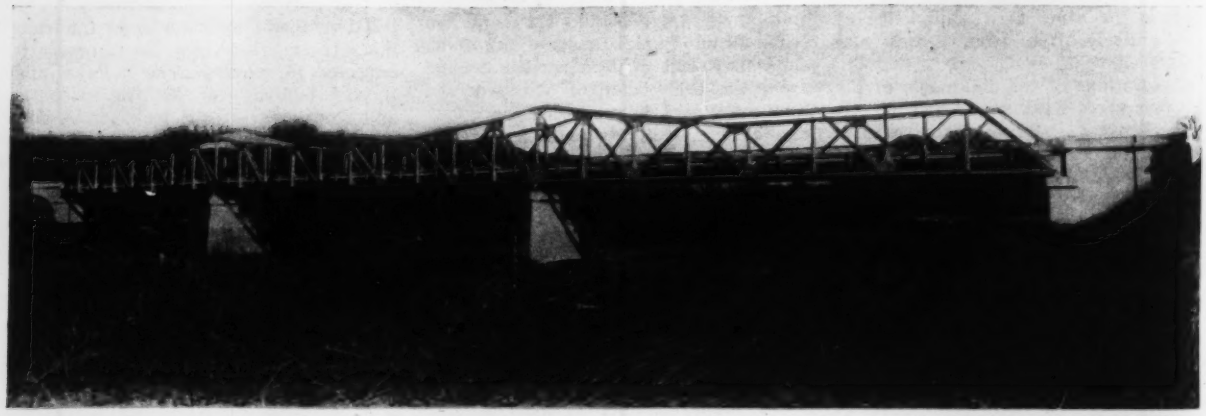
Each continuous group is fixed at a pier near the center of the unit, the other reactions being supported upon rockers or rollers and in many cases upon a single roller, of diameter as great as 12 in. The number of lines of girders or rolled beams selected for a given bridge is determined solely from studies of economy. In general the number is from three to five, although there has been designed, but not yet built, a 24-ft. roadway with only two lines of girders; on this structure, the stringers are also continuous over the floorbeams.

The floorbeams of the trusses and the diaphragms of the beam spans, located over the piers and abutments, have been designed for jacking the full dead load, including the floor slab, to permit the adjustment of reactions in case of settlement of foundations or future alterations.

The concrete floors of these bridges have been designed for H-15 and H-20 loadings, with no provisions for increase in live load, while the stringers, floorbeams, girders and trusses have been designed with provisions for a 100 per cent increase in live load, using one-third higher combined working stresses for this increased loading. The use of lighter floor loads than truss loads is predicated on the belief that higher concentrated loads are not to be expected but that higher total loads are a distinct possibility. The unit stresses used follow very closely those of the American Association of State Highway Officials. The floor slabs of the steel deck-girder spans are usually supported directly on the girders, but occasionally it is found economical to use a floorbeam and stringer system, which plan is invariably adopted in the case of truss spans.

In the design it is considered that the girders will probably have their maximum load with the full width of roadway loaded. However, on the wider bridges rather heavy diaphragms are used to take care of one-sided concen-

TWO TYPES OF CONTINUOUS BRIDGES used on Kansas highways. The low-truss bridge, with overhead bracing only at the piers, crosses Spring River in spans of 160-200-160 ft. The girder bridge over the Solomon River has five spans of 70-100-110-100-70 ft.



tration of load; no consideration is given to the added stiffness of adjoining spans in the design of the girders.

Structural silicon steel has been used for main girders in a few cases, such as the viaduct at Wichita (*ENR*, Oct. 18, 1934) and an almost identical viaduct that is being constructed at Hutchinson. Rolled beams of medium carbon steel are used for all floor systems and for the web members of the low-truss spans.

Continuous low trusses

The first continuous low-truss bridge that was built spans the Neosho River on U. S. Highway 75 near Burlington. It consists of a 128-160-128-ft. Warren truss unit of 24-ft. roadway width, flanked on one end by a 60-ft. simple beam span and on the other with two units each of three 70-ft. continuous-beam spans. The structure is founded upon shale. The depth of trusses is 14 ft. except over the main piers at points of maximum negative moment, where it is increased to 20 ft. Overhead struts are provided over each main pier, to provide general rigidity, although they are not theoretically required. The floor slab of the approach spans is supported directly upon five 36-in., 150-lb., wide-flange beams.

The longest span continuous low-truss bridge in the state is across Spring River on State Highway 96 in the southeastern part of the state. It consists of a 160-200-160-ft. unit similar except for span lengths, to the truss group of the Burlington bridge. Three overhead struts and two panels of upper lateral bracing are provided over each of the two main piers, but the structure is designed essentially as a low truss, and, as in the case of the Burlington bridge, upper laterals or struts are not theoretically required. The depth of trusses is 17½ ft. except over the main piers, where it is 25 ft.

Continuous plate girders

The bridge over the Solomon River on U. S. Highway 40N is a five-span deck-plate-girder unit, the spans being 70-100-110-100-70 ft. There are three girders spaced 10 ft. on centers, with an 8½-in. reinforced-concrete floor resting directly on the girders; the field splices in the girders are placed at the points of zero dead-load moment. There are no intermediate floor expansion joints in the entire 450-ft. length of the group. The abutments are of the open type with footings resting on piling; also all of the pier footings rest on piling except two that are founded on shale.

The design for a continuous deck-plate-girder unit constructed over Turkey Creek in the MKT-Frisco Viaduct on the Seventh St. Trafficway development in Kansas City, Kan. (*ENR*, Oct. 19, 1933) is unusual because of the large ratio of the 123-ft. length of the main span to the 52-ft. length of each end span, which resulted from clearance restrictions. In order to eliminate the possibility of the end spans lifting off their supports due to live loads in the main span, the rockers at the ends of the side spans were not placed until the concrete floor in the main span had been placed and the resulting upward deflection of about 1 in. had occurred at the ends of the group. The rockers at the ends of the group were then placed and shimmed up until a snug fit was obtained. Care was taken in placing these shims so that no appreciable reaction was obtained until the concrete floor on the side spans had been placed. This introduced enough dead-load reaction at the ends of the side spans to insure positive reactions under all conditions of loading, the spans acting as continuous for live load and only part of the dead load.

This is a 42-ft. roadway bridge designed for H-20 loading; there is a 5-ft. sidewalk on one side. Five girders are used in the continuous unit, spaced 9 ft. 10 in. on centers. The girders in the main span are of constant 6-ft. depth, while those in the side spans are haunched and have a depth of 3 ft. at the outer ends.

Rolled-beam bridges

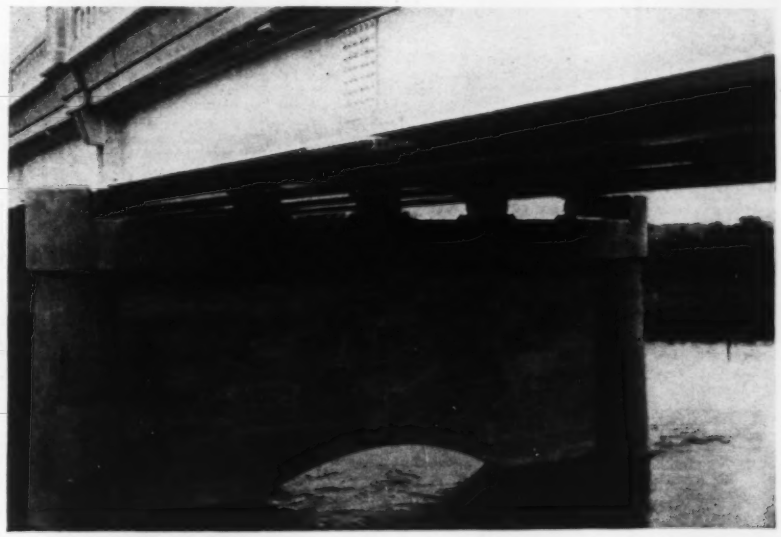
The facility with which rolled-beam continuous bridges have been adapted to various requirements is illustrated in the accompanying tabulation of span arrangements and lengths. There has been constructed at least one unit of each of those listed. The size of the rolled sections in these units has ranged from a 27-in. 85-lb. beam to a 36-in. 230-lb. beam.

The longest spans constructed of rolled beams are the 88-103-88-ft. units built across the Solomon River on State Highway 1. Across the Saline River on U. S. Highway 81, five lines of 36-in. 230-lb. rolled beams, 5 ft. 10 in. c. to c. are used.

Design methods used

Various methods of design and combinations of several methods have been used in Kansas, including the method of redundant reactions, the method of three moments, the method of moment distribution, the method of conjugate points, the slope deflection method, the method of column analogy and the moment-area method. If a person is familiar with all of these methods of design, there is a slight advantage in choice of method for special cases.

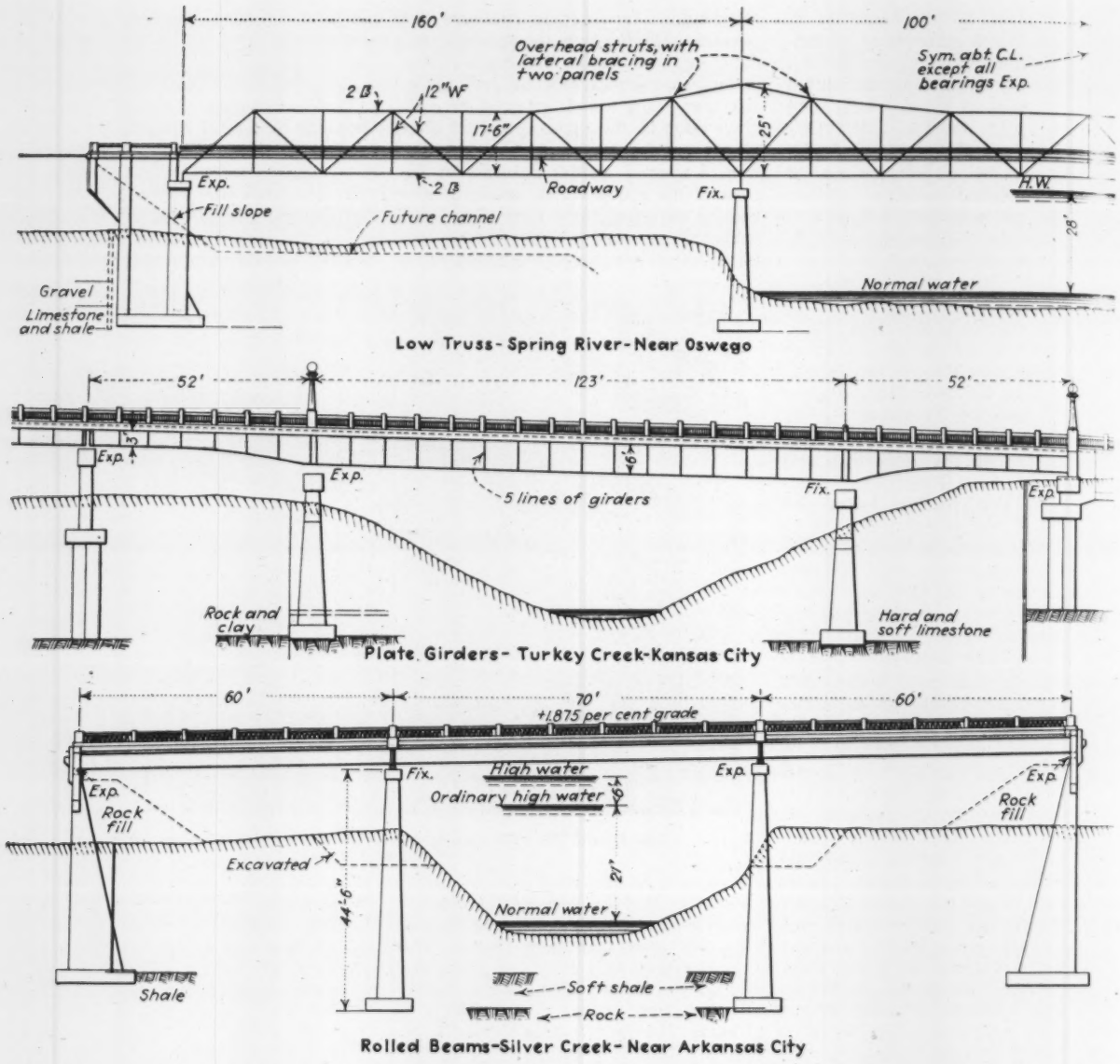
In the design of continuous girders of less than four spans having a constant moment of inertia, all of the previously mentioned methods can be considered equally good. In units having more than four spans and a varying moment of inertia, it seems more convenient for most designers to use column analogy to get the elastic properties of the beam, and then to solve for the moments, reactions and shears in the whole continuous unit by the graphical method of conjugate points or by the analytical method of moment distribution. In most instances Kansas structures have been designed by one method and checked by some other method. The designing and the checking of the continuous trusses



LARGE-DIAMETER rollers are used for bearings under some of the continuous rolled-beam bridges.

CONTINUOUS-BEAM BRIDGES AS USED ON HIGHWAY SYSTEM

Type of Unit	Span Lengths C. to C. Bearings
2-span	80–80
3-span	45–56–45
	55–65–55
	60–70–60
	70–80–70
	88–103–88
4-span	52–62–62–52
	47–72–72–47
	65–80–80–65
	71–71–71–71
6-span	45–52–56–56–52–45
7-span	45–52–56–56–56–52–45



were both accomplished by the method of redundant reactions.

Erection

Both the Burlington and Spring River trusses were erected on pile bents placed at every second panel point and bearing directly on the bottom chords, although it would have been possible to erect the center spans by the cantilever method if flood hazards had been serious.

The main pier reactions of the truss bridges (dead load of steel only) were correctly adjusted by means of hydraulic jacks, all eight reactions of the entire unit having been previously weighed by the same method. For the Burlington bridge little adjustment was necessary, but for the Spring River Bridge the intermediate bearings had to be raised about 0.1 ft. to obtain the proper ratio of reactions. The adjustment of the reactions in these cases was apparently accomplished with complete success that

THE FIFTY continuous bridges that were built by the Kansas highway department are of three types—rolled beams, plate girders and low trusses.

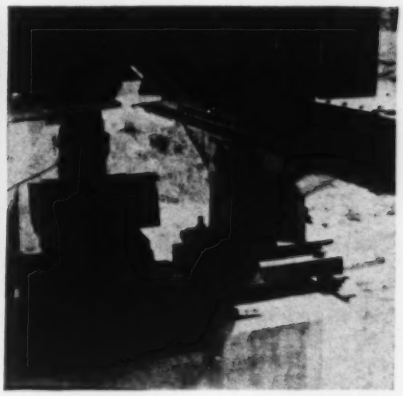
justified the small additional expense that was incurred. The error in ratio of the end reactions to the intermediate reactions that it was found necessary to correct in the Spring River Bridge, is attributed to the probable settlement of the falsework under the side spans after they were riveted and before the members over the intermediate piers were riveted.

No adjustment of the reactions has been considered necessary on any of the deck girder and beam bridges (except the Turkey Creek spans, previously mentioned) because the beams and girders are comparatively limber and because the girders are spliced at the points of zero moment under dead load of the steel alone which automatically eliminates the possibility of unknown moment or shear being locked up in them.

Although all of these continuous bridges have been founded upon shale rock or very good pile foundations, it is not considered of very great importance if the supports do have some slight unequal settlement. This is indicated by calculations and is borne out by the information obtained in adjusting the reactions of the continuous trusses. A small settlement of any support does not change the amount of the reactions, moments or shears to any great extent.

Economy considerations

As for the better economy of these continuous-type bridges over simple determinate spans, there are contributing items that are not directly caused by any of the stress features in the superstructure. Among these are the elimination of expansion joints, the reduction in the number of rockers and bolsters, and a very desirable reduction in de-



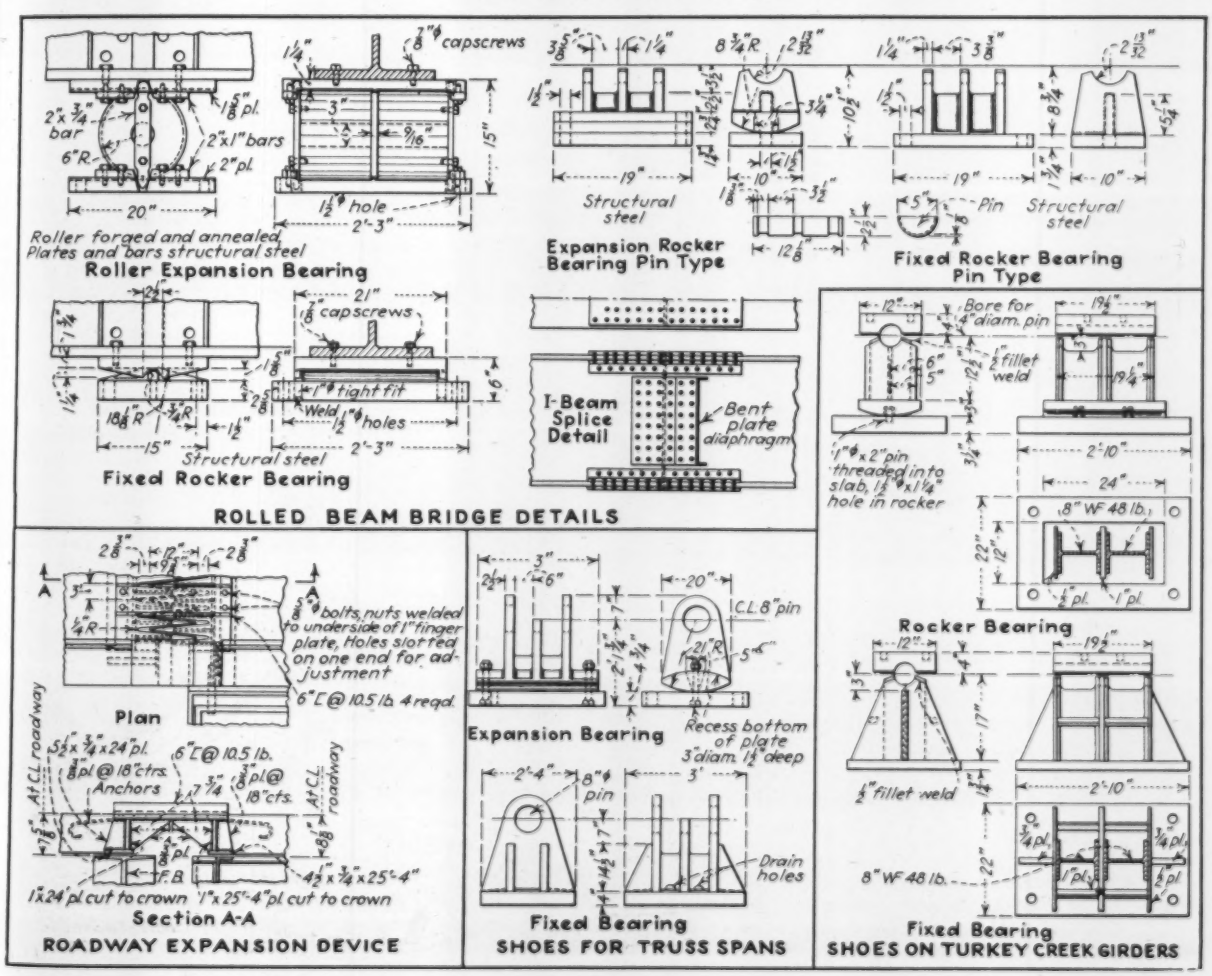
ADJUSTING pier reactions of Spring River truss bridge by means of hydraulic jacks. View at right, a close-up of jack and shoe, shows bearing plate in its adjusted position before grouting.

flections of about 50 per cent, which facilitates shallow deck construction. There is also a reduction in the size of the pier caps, some reduction in the number of end floorbeams, cross-frames or diaphragms, and the opportunity of increasing the economical span length that can be used with rolled beams, built-up plate girders and low trusses. Although, under double live-load conditions there are slightly more reversals of stress in the members of the low continuous trusses than in simple high or low trusses, it is not considered that there is much loss of economy, if any, from this fact. The members of the low trusses are relatively short and in all cases are constructed of rolled sections, which are almost equally good in either tension or compression; the Warren type trusses were selected mainly for this reason.

If, for the Burlington bridge, simple spans of the same length as those of the continuous-type structure had been used, the amount of structural steel would have been 1,034,000 lb., against the 935,000 lb. actually required. This is a direct saving in structural steel of over 10 per cent and with the additional savings, as mentioned previously, amounted to $8,000 in favor of the continuous design.

A comparative design for the Solomon River bridge near Osborne, consisting of five 90-ft.-span simple low

BEARING SHOES and expansion details used on various types of continuous bridges.



trusses, required 490,000 lb. of steel. The continuous girder structure that was constructed had a structural-steel weight of 395,000 lb., amounting to a saving in this case of 19 per cent in structural steel and a total saving of about $5,500.

Service experience

The first of these continuous bridges was built about four years ago. During this time no important difficulties have been experienced with the structures in service. Some of them have shown a tendency to crack at the curb over the piers and occasionally at unexpected places, but these cracks are very small and are not believed to be harmful. There is no evidence of cracking in the floor slab or any other indication of trouble due to lateral-load distribution. This may be because the maximum calculated loads have never occurred, but it is believed that the use of the rather heavy diaphragms combined with the small deflections of the girders relative to one another is adequate insurance against any bad results from lateral distribution. Checked dead-load deflections of all bridges have been very close to calculated deflections.

No trouble has been experienced with the rollers at the piers, although they do not move as much as might be expected.

These continuous bridges were designed in the bridge design department of the Kansas highway commission, with the exception of the MKT-Frisco viaduct on the Seventh St. Trafficway, for which the firm of Sverdrup & Parcel, of St. Louis, was retained by Kansas City, Kan., to prepare plans before the project was adopted by the state highway commission. All construction work was under the engineering supervision of the state highway commission.

French Build High Dam in Narrow Limestone Canyon

Sautet Dam in the French Alps is a constant-angle arch structure 414 ft. high backed by a mass of lean concrete to buttress the canyon walls—Elaborate grouting insures tightness and stability of the abutments, which are of Jurassic limestone

By Robert A. Sutherland
Designing Engineer With Lars Jorgensen, Consulting Engineer, Berkeley, Calif.

SAUTET DAM on the Drac River, one of the headwaters of the Rhone River in France, is the second highest dam in the world and by far the highest ever built in a limestone formation. A brief description of it was given under the head of "Engineering in Foreign Countries" in the May 30 issue of *Engineering News-Record*, p. 770. Because the dam is unusual both in design and in location, the following fuller description of it is given. Some drawings and data are reproduced here to make this description complete.

The Sautet Gorge is a magnificent canyon about 600 ft. deep and over half a mile long on the Drac River, a tributary of the Isère, which in turn is a tributary of the Rhone. It is situated about 45 miles from Grenoble, which is at the confluence of the Drac and Isère rivers.

The gorge is extremely narrow, and it was only in 1921 that the lower portion was first thoroughly explored. It was then realized that here was an unusual site for an economical dam capable of creating a storage of 106,000 acre-ft., which would serve as a regulating storage for eleven power plants, of which seven are already completed or under construction and four are yet to be developed. This dam, therefore, is the keystone to a structure of power development on the river system of the Drac and Drac-Romanche.

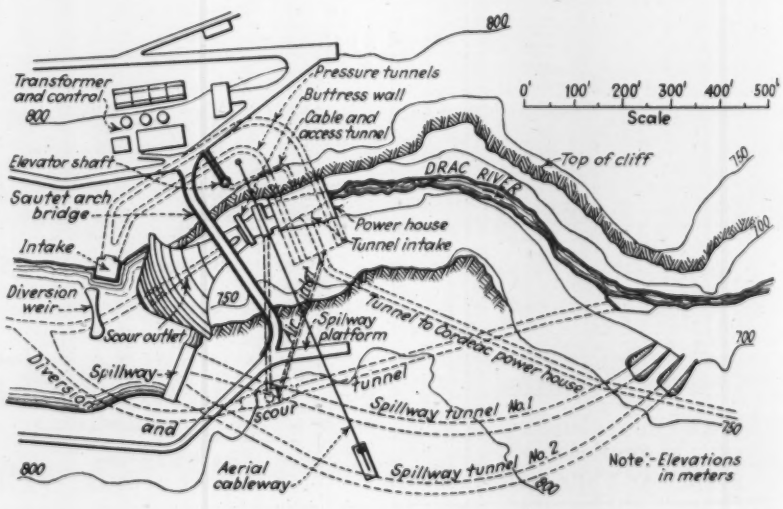
The existence of an excellent dam and reservoir site near the head of a river, with the possibility of regulating the flow for a number of downstream power plants, provides a great incentive to overcome any technical objections that there may be to the construction of a dam at that point. In the case of the Sautet Dam, the technical objections lay in the fact of the site being in a limestone gorge with overhanging sides. The means taken to overcome these objections through strengthening the canyon walls, together with provision for quick emptying of the reservoir and for further grouting if necessary, will be of interest to American engineers.

Rainfall and runoff

The Drac catchment is situated in the southern part of the French Alps, and has a total area of about 1,400 square miles, of which about 410 are above the Sautet Gorge. The highest points of the catchment reach an elevation of 12,000 ft., but the gorge itself is situated a little over 2,000 ft. above sea level. The average rainfall on the catchment is about 60 in.; the mean flow at the gorge varies from 600 sec-ft. at low water in January to 2,500 sec.-ft. during the melting of the snows. The average annual flow is 1,170 sec.-ft. The greatest flood of record is 28,000 sec.-ft., and spillway capacity has been provided for double this amount, which would occur, according to Fuller's formula, about once in 1,000 years.

Of the total capacity of the reservoir, 106,000 acre-ft., the lower 24,000 acre-ft. has been reserved for debris. At the prevailing rate of deposition, this will suffice for about 100 years. The upper 82,000 acre-ft. are intended for stream regulation, which will of course benefit all power plants below. This volume of water represents a reserve of energy of

FIG. 1—THE DAM is located at a narrow point in the gorge of the Drac River. Below it is the power plant partly in a gallery and partly in a building spanning the canyon from wall to wall.



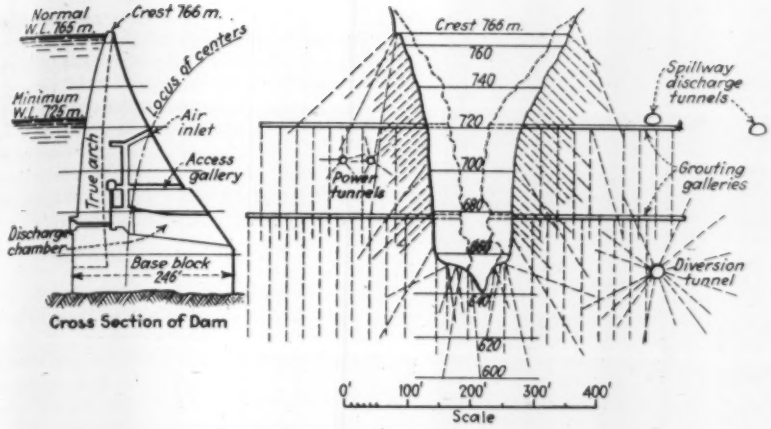


FIG. 2—MAXIMUM CROSS-SECTION of the dam and a section through the canyon walls on the line of the arch, showing the location of the grouting galleries and of the grout holes. The dam is a constant-angle arch backed by a mass of lean concrete giving the structure a form similar to that of a gravity dam.

about 110,000,000 kilowatt-hours when passed through all the plants constructed or to be constructed.

The total plant capacity installed or to be installed in plants above the Drac-Isère confluence is 331,000 kw., and the estimated gross annual output about 1 and 1/3 billion kw.-hr.

Site found to be satisfactory

Preliminary studies and examinations of the site involved the driving of about 1,650 ft. of exploratory tunnels, and the construction of a network of ladderways and gangways bracketed out from the almost vertical sides of the gorge. This preliminary examination proved the site to consist of a perfectly sound and compact limestone formation without faults or other imperfections, and capable of taking safely the thrust of a high arch dam.

The original plans for the dam comprised a comparatively slender constant-angle arch, followed by a smaller arch downstream that would reduce the effective head on the upper dam materially. However, construction of the lower dam was abandoned after diversion of the river made possible an examination of the gorge. This examination revealed a considerable extent of overhanging cliff, and to ensure absolute safety from possible falls of overhanging rock anywhere in the vicinity of the dam it was decided to back the thin arch on the downstream side with a solid mass of lean concrete, the main object of which is to support and buttress the canyon sides.

A small vertical buttress wall also supports a badly overhanging section of gorge side between the dam and the power house.

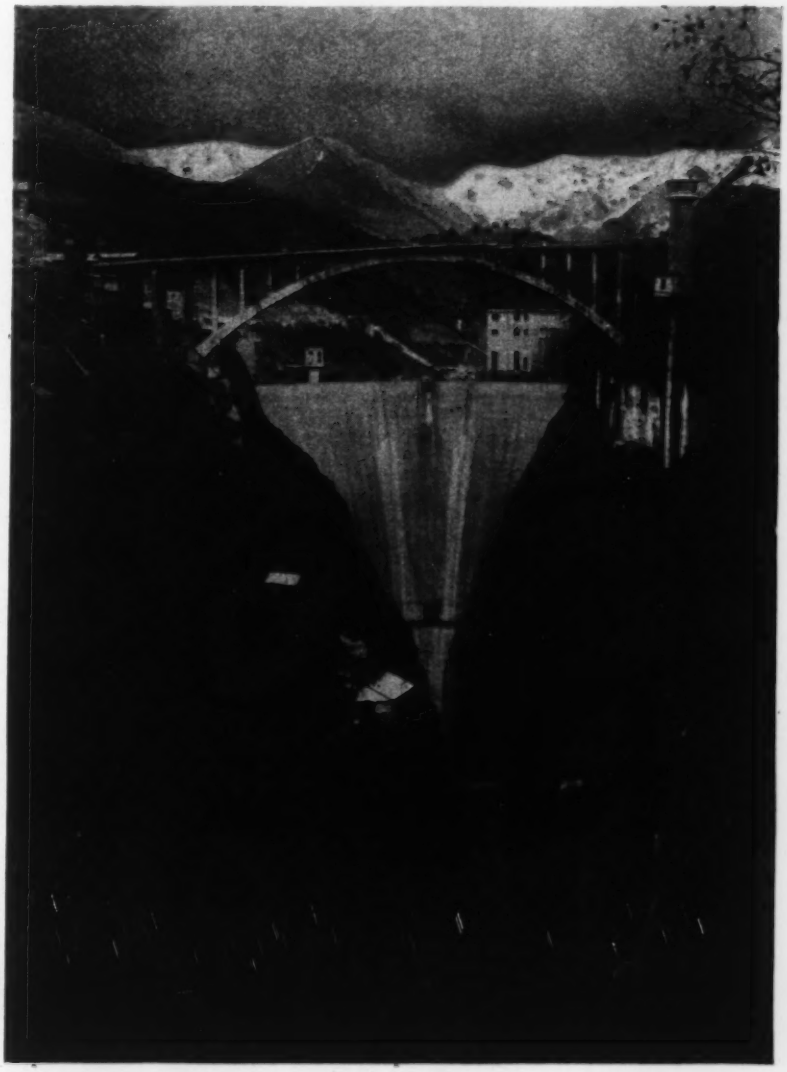
The cross-section of the dam approaches that of a gravity dam, although the water load is taken mainly through arch action by the comparatively slender arch section shown in the accompanying drawing. This division of the load results automatically from the higher modulus of elasticity of the concrete in the true arch.

Only at the abutments of the true arch were the side walls excavated to a radial bearing surface. The downstream portion of the dam represents merely filling material.

The total height of the dam is 414 ft., of which 98 ft. consists of a base block on which the arch proper rests. The crest length of the dam is only 263 ft. The dam above the base block was built in three sections, separated by two vertical contraction joints about 4 in. wide running right through the dam and afterward filled with concrete.

The total volume of the dam is 130,400 cu.yd. made up as follows: base block 35,900, arch proper 45,600, and backing 48,900 cu.yd. The total storage in the reservoir per cubic yard of concrete in the dam is about 0.81 acre-ft., thus making this an economical structure,

FIG. 3—SAUTET DAM rises 414 ft. above the floor of a canyon whose walls at the narrowest point are less than 50 ft. apart. In the gorge below the dam is a buttress between the overhanging cliffs, and in the foreground is the roof of the power house.



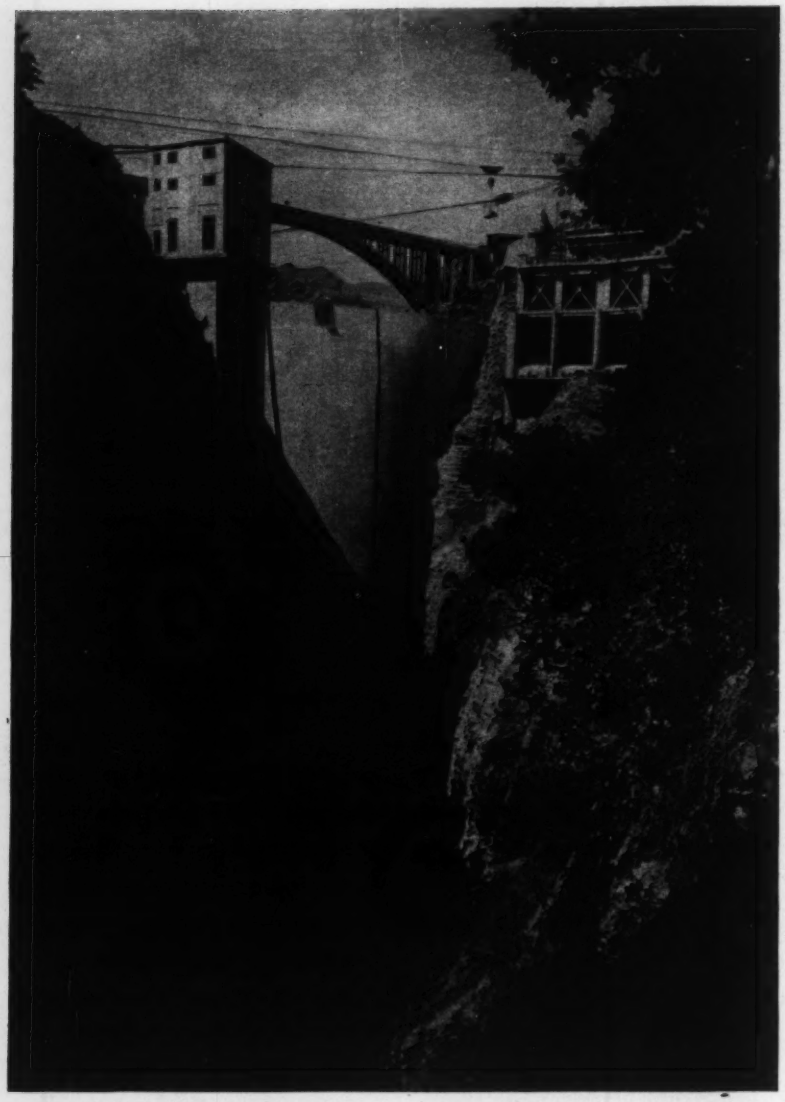


FIG. 4—UPSTREAM FACE of the dam as it neared completion. The spillway tunnel openings are at the right, and at the left is the intake structure.

although not approaching in this regard many of the storages in this country, e.g. Lake Cushman, for which the storage is over 5 acre-ft. per cubic yard of dam concrete.

Extensive grouting undertaken

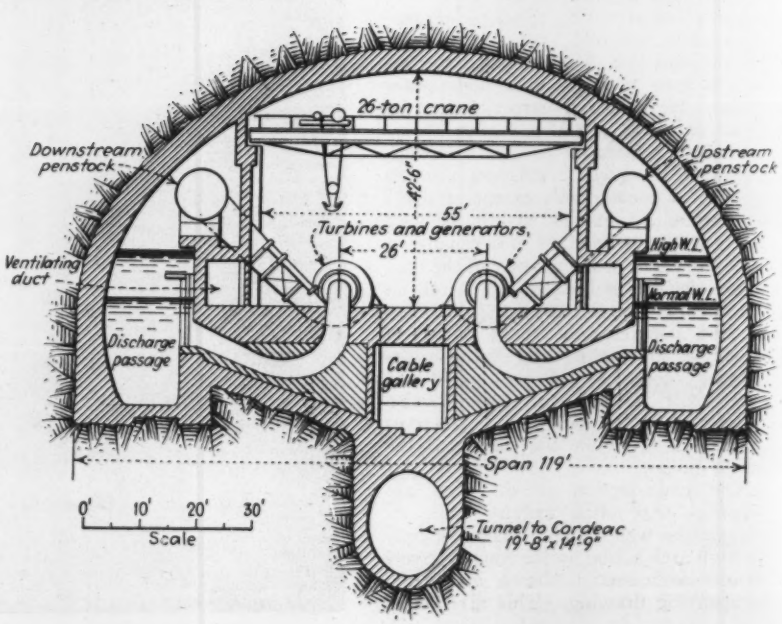
Tightness of the rocky sides and bottom of the canyon was assured by a very thorough grouting program, particularly in the line of the arch proper. In all, about 20,000 lin.-ft. of grout holes were drilled and grouted, absorbing about 3,000 tons of cement, under pressures up to as much as 500 lb. per sq.in. Part of this grouting was done from tunnels that have been left open in order that further grouting may be done at any future date, should such appear at any time advisable.

During construction of the dam, a very complete system of indicating devices was installed, including 20 points of measurement of hydrostatic pressure, 43 strain detectors of the acoustic type (some of which are also adapted for temperature measurement), 19 temperature indicators, and 4 deflection meters of the sensitive-pendulum type. The latter are installed on a vertical line at the center of the dam. All the connections from these devices are brought to a central chamber in the dam.

Several outlets provided

Floods are discharged by means of a simple weir of ogee shape, which opens directly into two tunnels of 370-sq.ft. cross-section each. These tunnels are over 1,000 ft. long and discharge into the river well below the power plant. The tunnels are so arranged that the water will flow with a free surface, and the maximum velocity is estimated to be about 80 ft. per sec. Three other tunnels are provided, one for the emptying of the reservoir (having been used during the construction period for diversion) and two for the supply of the power plant. The diameter of these latter reduces from 14 ft. 3 in. to 13 ft. 3 in., and at the lower end they are lined with 14-in. minimum thickness of reinforced concrete. The capacity is 1,590 sec.-ft. each. There are also two outlets through the lower portion of the dam, fitted at the downstream end with jet dispersers to dissipate the energy of the jet. These are provided to bypass water for lower plants, in the event this plant is partly or wholly shut down. It is estimated that by the use of all the means of discharge mentioned, the reser-

FIG. 5—SECTION OF THE POWER HOUSE, showing the position of the turbines in the 119-ft.-wide gallery cut in the canyon wall. As shown in plan in Fig. 1, the central room of the power house is carried across the canyon on a concrete arch that also carries the penstock lines.



voir can be completely emptied in case of need in nine to thirteen days, depending on the discharge of the river at the time. This provision is desirable to allow measures to be taken in the event that in spite of all precautions unforeseen leakage or fall of rock should occur. Several dry tunnels are provided for access and for air inlets to the diversion control gates.

Problems of location

The location of the power plant presented special problems on account of the extremely narrow width of the lower part of the gorge. It is located at right angles to the stream, partly in a gallery excavated into the right bank of the gorge, and partly in a hollow reinforced-concrete structure spanning the gorge. It contains six double-runner Francis turbine or generator units, each of 13,250 kva., running at 500 r.p.m. under a head that varies from 174 to 305 ft., depending on the lake level. The number and size of units were fixed by limitations of space and of weight of individual pieces. The turbines are designed for 13,000 hp. each at the mean head of 240 ft. At higher heads the guide-vane opening is restricted to limit the flow to 530 sec.-ft. per set, while at lower heads the power developed decreases to a minimum of 7,000 hp. Cooling of the generators is by closed air circuit, provided with water-cooled pipes.

Water discharged from the turbines passes through side channels into a tail pond directly below the dam from which it flows into a tunnel supplying a lower power station, named Cordeac. The intake of the Cordeac tunnel is in the canyon directly below the power house. The tunnel passes back into the canyon wall under the center of the gallery excavated for the power house (Fig. 5).

It was originally intended to develop the entire head of about 610 ft. at the latter plant, but this plan was rejected to avoid the use of a pressure tunnel over 3 miles long under a high head, also to reduce initial capital expenditure.

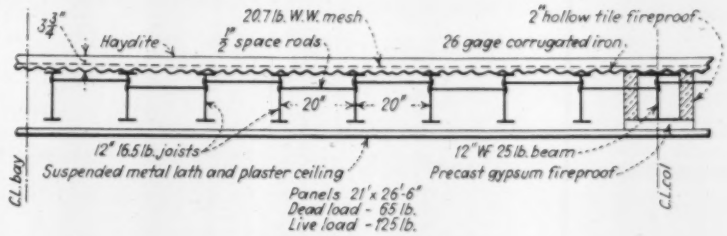
Limitations of space required that the transformers and control equipment be located above the dam at the side of the gorge. The generators are connected thereto by cables running through a gallery and up the access shaft. This shaft is also provided with two elevators for passengers and light material. Heavy material can be delivered to a platform just upstream of the power house by means of an aerial cableway, which is a permanent installation.

The entire plant was designed and constructed, with financial assistance and supervision of the French government, by the "Forces Motrices Bonne et Drac," from whom much of this information was obtained. Acknowledgment is also made to M. André Coyne, chief engineer, Ponts et Chaussées, for photographs and information.

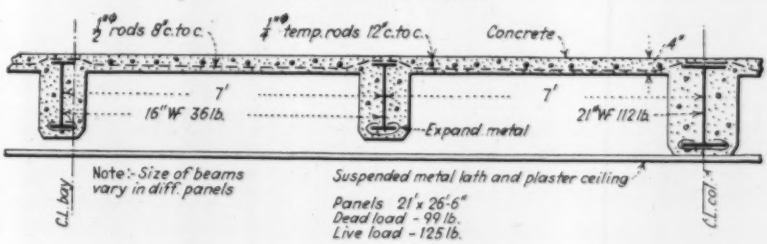
Costs of Two Types of Floor Construction Compared

By Dwight A. Dougherty
Greencastle, Ind.

DURING recent years many new types of floor construction have come into use. One of the latest developments has been the junior beam-corrugated iron type. Because of its comparatively recent development, little is known concerning its cost. Since economy is often the deciding factor in the selection of a floor, the following job-cost comparison in which it proves 23.6 per cent cheaper than the beam-and-



Junior Beam Construction



Beam and Girder Construction

TYPES OF FLOOR CONSTRUCTION whose costs as they were recorded on the same job are compared.

COMPARATIVE FLOOR COSTS PER SQUARE FOOT

	Labor and Material Excl. of Steel		Struct. Steel		Total	
	Net*	Overall†	Net	Overall	Net	Overall
Junior Beam—Corr. Iron	$0.444	0.040	0.253	0.228	0.696	0.629
Beam—Girder	0.776	0.691	0.137	0.121	0.912	0.813

*Openings figured out—14,460 sq. ft. for junior beam and 14,253 for beam and girder.
†Based on 16,000 sq. ft. for both types of floor.

UNIT AND TOTAL COSTS, LABOR AND MATERIAL

Junior Beam—Corrugated Iron—Haydite Concrete Fill
16,000 Sq.Ft. Overall—14,460 Sq.Ft. Net

	Am't	Man.-Hr.	Cost Per Man-Hr.	Cost Per Sq.Ft.	Sq.Ft. Per Man-Hr.
Labor—screeds and strike-offs	$231.33	313	.74	.016	46.2
Labor—reinforcing steel mesh	53.50	53½	1.00	.003	270.0
Labor—prep. and placing Haydite	121.27	243½	.50	.008	59.5
Labor—laying corrugated sheets	109.92	101	1.08	.007	143.0
Totals	515.82	711	.725	.035	20.4
Material:					
Corrugated sheets	$502.50	....	....	.034	.....
Tack weld (contract)	47.00	....	....	.003	.....
Wire mesh	125.00	....	....	.008	.....
108½ cu.yd. ready-mixed Haydite	775.00	....	....	.053	.....
Fireproofing—hollow tile (contract)	2,032.00	....	....	.143	.....
Suspended ceiling—lath and plaster (contract)	2,414.82	....	....	.167	.....
	$5,896.32	....	....	.408	.....

Beam-and-Girder Concrete Floor Arch
16,000 Sq.Ft. Overall—14,253 Sq.Ft. Net

	Am't	Man-Hr.	Cost Per Man-Hr.	Cost Per Sq.Ft.	Sq.Ft. Per Man-Hr.
Labor—screeds and forms (build, strip and clean)	$3,279.00	5,298	0.62	0.23	2.69
Labor—reinforcing steel	306.00	357	0.857	0.021	39.9
Labor—prep. and placing concrete	834.00	1,529	0.551	0.058	9.3
Totals	$4,419.00	7,184	0.616	0.31	1.98
Material:					
Forms	$1,400.00	....	....	0.10	.....
Reinforcing steel, 10½ tons	525.00	....	....	0.036	.....
400 cu.yd. ready-mixed concrete	2,300.00	....	....	0.161	.....
Suspended ceiling—lath and plaster (contract)	2,415.00	....	....	0.169	.....
	$6,640.00	....	....	$0.466	....

girder type, should be of value to a designer contemplating the use of this type of floor construction, and of particular value to a contractor figuring a job on which it is specified.

The two floors under consideration were on the same job and were constructed under practically the same conditions. Although the structural steel in a building is usually figured separately, the cost here is given both with and without the structural steel added. The cost of the structural steel chargeable to the arch is for the intermediate beams or joists and junior beams only and not for the columns or the beams on the column lines, as they are considered a part of the framing of the building.

It will be noted that the topping, or floor finish, is not included in these costs. This item is omitted in order that the total cost data may be more readily applicable to floors carrying different finishes from those under consideration here.

These floors were designed for a department store erected in the Middle West during the past year. The scale of wages per hour for the trades engaged in the construction of the arches were: carpenters, $0.85; cement finishers, $0.75; iron workers, $0.90 for reinforcing steel and $1.12½ for corrugated sheets and structural steel; bricklayers, $1.25; hod carriers, $0.72½; plasterers, $1.20; hod carriers, $0.80; and common labor $0.40.

As shown in the figures on p. 709, the total cost of labor and material, exclusive of structural steel, for the junior beam corrugated iron type of floor was $6,412.14. Structural steel cost was $3,650.33 (63.75 tons at $57.26 per ton), making the total cost $10,062.47. Comparable figures for the beam-and-girder floor were $11,059, exclusive of steel, $1,941 for steel (33.9 tons at $57.26 per ton) and $13,000 total. The unit values based on these totals are given in the second table.

New Deep-Water Port at Cartagena, Colombia

Old Cartagena is assuming rank among modern South American seaports with the construction of a bulkhead wharf and piers of concrete and a dredged basin for ocean and river steamships

By R. H. Cady
Frederick Snare Corp., New York City

THE PORT of Cartagena on the north coast of Colombia shares the export and import business of the country with Port Colombia, on the same coast farther to the east, and Buenaventura on the Pacific Coast. Although of secondary commercial importance in the past, Cartagena, due to recent important developments, is now rapidly becoming the leading port of Colombia. Its natural harbor is one of the finest on the continent.

The development of Cartagena as a commercial port received little attention until late in the past century. In 1884 the first wharf of any importance was built for shallow-draft boats. In 1891 a small wooden pier, known as the Machina and capable of docking ocean ships, was built. In 1894 a railroad about 65 miles long was constructed to connect Cartagena with the town of Calamar, located on the Magdalena River about 65 miles from its mouth, thus supplementing the inland canal known as the Dique, which connected the same two points.

The Magdalena River, running north through the heart of Colombia, is the main artery of traffic of the country and, except for plane service, no other means of communication exists between the interior and the north coast. The Dique, partly a natural and partly an artificial channel running through the region of swamps and lakes between the Magdalena and the harbor of Cartagena, was the sole means of communication between Cartagena and the interior river system of the Magdalena before the construction of the railroad. Until about 1923, when an extensive program of dredging and improvement of the Dique was begun, this waterway supported only an intermittent traffic of small boats. Unfortunately the dredging had to be stopped in 1929 before being completed. In the same year the entrance to Cartagena Harbor at Boca Chica had been improved by dredging a straight channel 40 ft. deep and 500 ft. wide so that the largest ships could safely enter the bay.

In the meantime the construction of another railroad, the Bolivar Central, was started and built a short distance from Cartagena, when lack of funds caused it to be abandoned. This line was intended to extend to the Cauca Valley, a tributary of the Magdalena and one of the richest portions of the country. Although for the present this project has no commercial importance, it has great possibilities for the future when it may be of especial importance to Cartagena as a port. Fig. 2 shows the several places and developments that have been mentioned.

Shortly after this state of development was reached, in August, 1930, the Machina pier above mentioned was destroyed by fire. This afforded the Colombian government, which had recently recovered the pier concessions, an opportunity to undertake studies for new and independent developments of the port along modern lines. After some preliminary study, Frederick Snare Corp. was engaged to examine the needs

FIG. 1—TWIN PIERS and quay wharf of concrete completed as part of development plan for the port of Cartagena, Colombia.

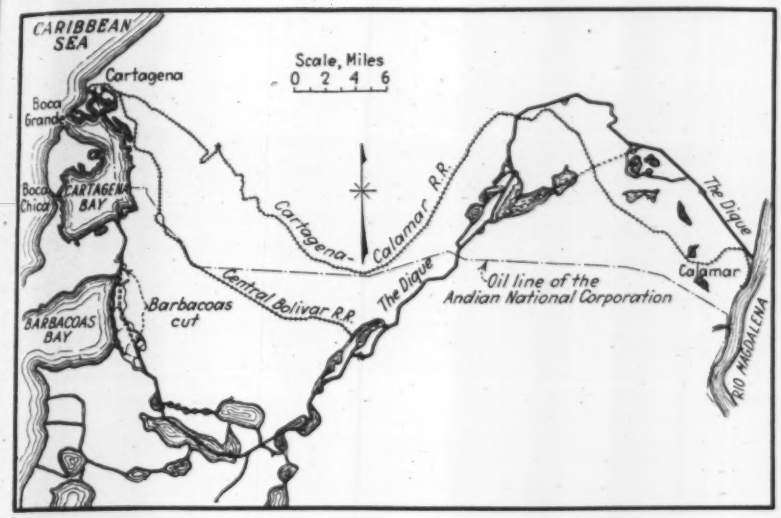


FIG. 2—CARTAGENA HARBOR and its rail and canal connections with the Magdalena River and the interior of Colombia.

FIG. 3—HARBOR ENTRANCES, new port works and other geographical features related to the construction operations at Cartagena.

of the port and plan the new construction required.

An unusual opportunity for improvement was found, since all the existing facilities, including the burned pier, the Dique steamer wharf, railroad connections, warehouses, etc., had been inadequate in capacity, widely separated, poorly located and in other ways disorganized to such an extent that the cost and delay in handling and rehandling cargo was excessive and the general service to local and foreign shippers was highly unsatisfactory. It was found possible to plan an entirely new terminal that would combine the various functions of the former facilities, and also to build it in such location that future expansion could be made without congestion of any sort.

Selection of site

A site combining practically all possible advantages was selected on the north shore of Manga Bay (Fig. 3). Its situation with respect to present and probable future growth of the city, highway and rail connections both present and future, accessibility to ocean and Dique boats, degree of shelter to shipping and room for future expansion were all practically ideal. Investigation showed the bottom to consist of sand mixed with broken coral overlying firm blue clay at about 40 ft. below sea level, a condition suitable for dredging to the required depths, with good support for pile foundations below. The required amount of dredging was found to be relatively small but sufficient to reclaim a swampy shore area (Figs. 4 and 5) that was more than adequate for the needs of the port. Since in Colombia the rights to the use of the actual shoreline are reserved to the nation, it was possible by this plan to avoid the purchase of any extensive properties for the terminal zone.

Based on the surveys and studies of the needs of the port, a report was made to the Colombian government in May, 1931, accompanied by general plans, estimates and detailed recommendations for a new terminal. This resulted finally in the award of a contract in April, 1932, for the construction and administration of the works recommended, described below. The contract price was fixed at $2,850,000 U. S. and the time of construction at 27 months; both price and time of construction were guaranteed by the contractor.

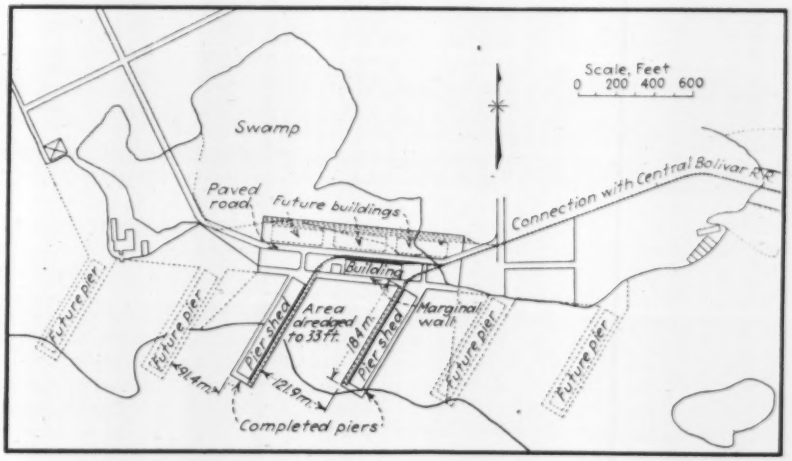


FIG. 4—PORT STRUCTURES as completed and planned for port development at Cartagena, Colombia.

Description of new port works

High humidity, high temperature and salt-laden trade winds make the atmospheric conditions at Cartagena peculiarly destructive. This, combined with the activity of the teredo and the termite, required the most permanent and resistant type of construction and resulted in extensive use of reinforced concrete asbestos roofing, etc., and special attention to details of design and protection of metallic materials. Due to the limited selection available in local concrete aggregates, conservative requirements for strength were specified, and the design was prepared accordingly, although in practice it was found possible to exceed specified strengths by as much as 80 per cent.

Bulkhead Wharf—A bulkhead wharf 1,065x33 ft., located so as to balance the dredging and fill (Figs. 4 and 5), serves to berth part of the shallow-draft Dique boats. Vertical and brace piles are 18x18 in. square, and sheetpiles retaining the fill are 14x18 in.; the piles vary from 50 to 70 ft. in length. The deck is designed to carry 750 lb. per sq.ft. live load in addition to a traveling gantry cargo crane and a future railroad track. The design also utilizes the resistance to uplift of the sheetpiles as well as a portion of the deckweight to aid the brace piles in resisting the pressure of the hydraulic fill. The construction throughout is reinforced concrete. A

FIG. 5—CARTAGENA port structures from the land side, showing the inland areas that were filled in by hydraulic dredge.

FIG. 6—AGGREGATE PLANT at Castillo Grande, showing stockpile formed by hydraulic dredging from harbor bottom.

clear cover of 3 in. of concrete is provided for the main reinforcing in piles and beams and 1½ in. in deck slabs. Six bags of cement per yard were used for deck concrete and 7½ bags for pile concrete. A pavement 4 in. thick of 8½-bag concrete, unreinforced, covers the top of the wharf, and emulsified asphalt was used as a waterproofing to the under surfaces of the wharf and to the pile surfaces above the water line. The fender system consists of a flexible arrangement of heavy native hardwood bolted to the concrete and creosoted pine piles, designed so that the piles take most of the wear and may be easily renewed.

Piers—Two piers, 130 ft. wide by 600 ft. long and 400 ft. apart (Figs. 1, 4 and 5), were constructed at an angle of about 60 deg. with the wharf. The total berthing space of the piers and wharf is about 3,280 ft. One side of each pier is available for either ocean ships or Dique boats, thus allowing some flexibility in the use of the berthing space, depending upon the widely fluctuating demand. The construction (Fig. 7) of the piers is in general similar to that of the wharf described above, except that some piles as long as 84 ft. were required.

Dredging and Fill—An approach channel 500 ft. wide was provided by widening and straightening the natural channel between Manzanillo shoal and Manzanillo Point (Fig. 3). This channel leads directly to the most westerly berth of the new piers. A depth of 39 ft. at low water (40 ft. at high tide) was provided in the approach channel and the most westerly berth; a 33-ft. depth at low tide was provided at the other pier berths and 15 ft. at the bulkhead wharf. About 500,000 cu.yd. of material, which consisted largely of broken finger coral and coral sand, were removed by a suction dredge and deposited in the swamp area (Fig. 5) behind the bulkhead wharf, creating about 40 acres of new land. Some difficulty was experienced with large lumps or boulders of coral. The fill produced was firm and self-draining.

Buildings—Each pier is equipped with a warehouse 570 ft. long and 80 ft. wide, clear span, and a similar building 300 ft. long is provided behind the wharf between the two piers. These are of steel-frame monitor-roof-type buildings with continuous side doors, steel sash, corrugated asbestos roof and concrete and walls. The asbestos roofing used is of the large (6-in.) corrugated type, red in color. A two-story office building about 50x50 ft., of similar type of construction, is provided for the customs officials and the administration of the terminal.

Cargo-Handling Equipment—Ocean ships are expected to use their own gear for loading and unloading, but special equipment is provided for the Dique boats and barges that have no gear of their own. On one side of each pier two electric cranes of 3-ton capacity are provided, traveling on semi-portal gantry frames. These are of the level-luffing rapid-action type and are believed to be the first of this type to be manufactured in the United States. Two similar cranes of full-portal type travel along the bulkhead wharf, and a fixed portal crane of 15-ton capacity is centrally located for handling heavy lifts to and from barges. A 25-ton floating derrick with a 90-ft. steel boom completes the heavy lift equipment.

Other floating equipment consists of a motor launch, a 100-hp. diesel tug and several steel lighters. For handling coffee, the principal export cargo, from Dique boats and barges to the warehouses the electric cranes are used when open barges are available, but main reliance is put on gasoline-powered elevators and conveyors which permit easy and rapid classification of the various lots of coffee as they are received in the warehouses. For handling coffee from storage to ocean ships, one-ton loads (about sixteen bags) are placed in rope slings on four-wheel trailers and taken in trains by a gasoline tractor to the ship's side where the ship's tackle hoist them directly aboard. These trains are frequently loaded in advance of arrival of the ocean ship, and this contributes greatly toward increased speed of dispatch.

Railway—A portion of the filled area is paved to facilitate transfer of cargo, and two concrete roads connect with the highways leading to the town. Track facilities are provided to serve all warehouses with provisions for considerable future expansion. A hydraulic-fill embankment and reinforced-concrete trestle crossing the end of Manga Bay provided the means for connecting the ter-

minal track work with the existing Cartagena-Calamar Railroad and the future Bolivar Central without cutting through valuable business or residential property of the city.

Dredging in the Dique—The contract for the new port works included 2,100,000 cu.yd. of dredging in the Dique. This consisted principally in the completion of Barbacoas cut, a new cutoff (Fig. 2) in the Dique route designed to avoid the open water passage of Barbacoas Bay, which has previously been a source of delay and danger to the Dique boats. The material removed by suction dredge varied from light silt to extremely heavy compacted clay.

Construction Methods—No suitable site near the location of the new piers was available for casting the concrete piles, for storage and for other purposes, and it was therefore decided to start operations at Castillo Grande (Fig. 3) where the concrete aggregates were stored. A temporary pier, office, warehouse, etc., were built at this point and also living quarters for the principal members of the contractor's staff. These consisted of cottages of temporary timber construction especially designed for the climate, with porches on all sides, high ventilated ceilings and ample cross-ventilation in all rooms. Independent water supply and sewerage systems were installed.

Concrete Aggregates—Concrete stone available from local quarries was found to be a soft and porous coral limestone, and the beach sand, normally used, was excessively fine. A much better source of aggregates was developed by dredging a relatively coarse sand gravel deposit near Castillo Grande. About 50 per cent of the material pumped was recovered by allowing the coarser material to drop out of holes burned in the bottom of the discharge line. The stored material was afterward put through a screening and washing plant, which permitted the selection of suitable sizes of sand and gravel. Additional fine material was wasted in the washwater, and the excess sand was discarded. About 2 cu.yd. of material deposited by the dredge were required for each yard of concrete finally produced.

A considerable quantity of shell was found in the gravel, but most of it was removed by allowing the gravel to fall in a thin sheet through the blast of an electrically driven air blower. After some experimentation an arrangement was devised that removed all the lightweight flat particles so that the only shell particles remaining in the gravel were relatively thick and dense. Although it was feared that the presence of shell would cause a decrease in strength with the aggregates thus prepared, no difficulty was experienced in maintaining an average strength 80 per cent in excess of that specified.

Casting and Driving Piles—Nearly 3,000 bearing, brace and sheetpiles were required for the job. These varied in length from 50 to 84 ft., and with the exception of a few of the longest were all cast and cured in Castillo Grande and then transported by lighter to the site of the work. The longest piles, over 76 ft., were required last and were cast on a portion of the completed pier deck. All piles were designed to be picked up and handled by bolts passing through the pile at two points. For this purpose galvanized pipe sleeves were cast in the piles, and extra reinforcing was used at the pick-up points, which were located 22/100 of the length of the pile from each end. For the amounts of steel used, this made the maximum steel stresses at points of support and center of the piles approximately equal. An allowance for impact in handling, equal to 25 per cent of the deadweight stresses, was provided in the design.

All piles were cast in solid stacks, cured for 28 days, then picked up by gantry crane, loaded to flat cars and transferred to lighters by dock crane, the two-point support being used throughout. Bearing piles were driven to 100-ton capacity (*Engineering News* formula) by steam hammer and were cut off at the desired level, exposing the pile reinforcing for bonding with the superstructure. The pile lengths had been determined by driving a number of full-size timber test piles and allowing a small excess length for cutoff. This proved satisfactory except for one low area that was missed by the test piles, where about 40 piles had to be spliced and redriven.

Time Required—The contract, which was dated April 28, 1932, was scheduled to be completed in 24 months and guaranteed to be completed three months later, or July 28, 1934. Active work was started in May, 1932, and the first concrete piles were poured at Castillo Grande in August of that year. Actual contract work was completed in December, 1933, less than 21 months after date of contract.

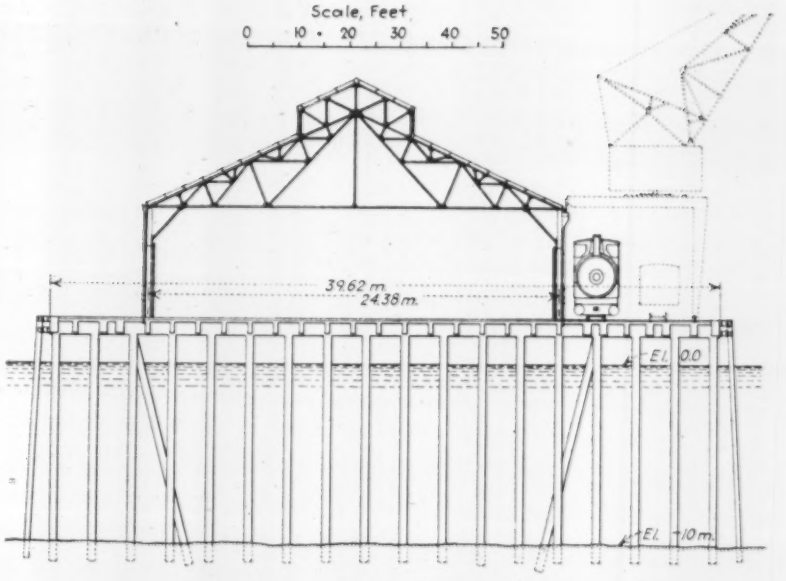


FIG. 7—CROSS-SECTION of pier and superstructure for Cartagena port works

Operation—As stated above, the construction contract also included the administration of the completed works. This was arranged in order to guarantee the loans made by the contractor and others toward the construction costs. No concession in the ordinary sense, however, was given the contractor, since it was provided that all earnings should be applied directly toward liquidation of the loans, the government having the right to take over the operation of the terminal at any time by full settlement of the unpaid balance, when funds are available from other sources. Although this imposed upon the contractor the unusual obligation of organizing the operation management together with the necessary tariffs, rules and regulations, this arrangement not only made possible the construction of the terminal during difficult financial times but made it possible for the government eventually to take over the terminal as a going concern of known financial value.

Contractor's Organization—For the contractor, G. P. Seeley, vice-president, had general charge. R. H. Cady, general representative in Colombia, had direct charge of preliminary studies, estimates and reports, negotiation of contracts, detailed design and construction, and organization of administration. R. R. Ellis, Jr., acted as assistant engineer throughout and local office manager during construction. W. J. Coleman was general construction superintendent with C. S. Canals and S. T. Barker assistants. All dredging work was under subcontract by Gahagan Construction Corp., F. R. Van Stone superintendent in charge. G. H. Birchall is now resident operation manager of the terminal.

New Viscosimeter for Bitumens Has Extended Range

Apparatus and procedure developed for a consistency determination that can be applied to all grades of bituminous road materials

By E. O. Rhodes, E. W. Volkmann and C. T. Barker
Technical Department, Koppers Products Company, Pittsburgh, Pa.

A PROCESS for determining viscosity that is applicable to all classes of bituminous-road materials is an important development. Such a process, including the invention of a new viscosimeter, is described in the following article. The new process makes notable claims for convenience, speed, accuracy and adaptability that will interest highway engineers and their bituminous laboratory workers. —EDITOR.

MANY GRADES of bituminous material are used in the construction and maintenance of bituminous roads and pavements. Whether derived from coal tar or from petroleum, they are classified principally according to their consistencies. These vary greatly from the low viscosities of light priming materials, almost as fluid as water, to the high consistencies of heavy binders, semi-solid at normal atmospheric temperatures.

There has been no single, commercial consistency determination suitable for the testing of all grades of bituminous road materials. In the United States consistencies usually are determined by and expressed in terms of Saybolt-Furol viscosity at 25, 50 or 100 deg. C.; Engler seconds or specific viscosity degrees at 40 or 50 deg. C.; float-test seconds at 32 or 50 deg. C.; and softening points by cube-in-water or ring-and-ball methods. In England and Europe some of the above-mentioned tests are used. In addition, consistencies are determined with the Tar Viscometer at 30, 35 and 40 deg. C.; with the Hutchinson tar tester (junior and senior models) at 25 and 30 deg. C.; and with the Kraemer-Sarnow softening-point apparatus.

Each of the above-mentioned tests is suitable for only a portion of the entire consistency range of bituminous road materials; the results of each are expressed in empirical units. The resulting confusion of Saybolt-Furol seconds, Engler degrees, float-test seconds, Tar Viscometer seconds, Hutchinson seconds and softening-point temperatures by cube-in-water, ring-and-ball and Kraemer-Sarnow methods makes comparisons difficult.

In a preceding paper (*ENR*, Oct. 19, 1933) one of the authors presented a consistency conversion chart for road tars. The preparation of that chart afforded an opportunity to study the precision and suitability of each of the test methods enumerated above for different grades of road tar. It was concluded, for the lower consistency ranges, that Engler specific viscosities at 40 and 50 deg. C. and, for the higher consistency ranges, float tests at 32 and 50 deg. C. were most satisfactory. The general acceptance and use of these two test methods for road tars was recommended, "at least until some future time when a single consistency determination is developed that can be used satisfactorily for all grades of tar-road materials."

Apparatus now developed

This possibility has now been accomplished. An apparatus (patents applied for) has been developed in the laboratories of the Koppers Products Co. by means of which the consistencies of all grades of road tar can be determined quickly and conveniently at a single temperature, within the range of atmospheric temperatures encountered under service conditions. All results are expressed in terms of the absolute unit, the *poise*, which may be defined as follows: If the space between two planes, 1 cm. apart and each 1 cm. square, is filled with a fluid and a tangential force of one dyne is required to move one of the planes 1 cm. in one second, the fluid has a viscosity of one poise. The dimension of viscosity (in

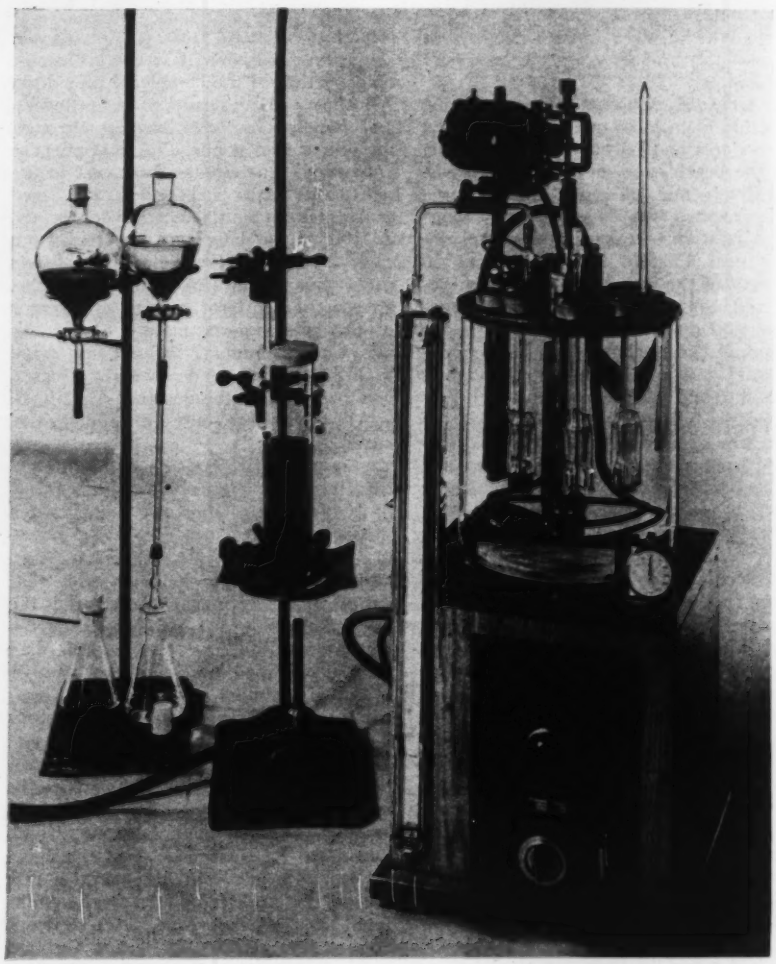


FIG. 1—VISCOSIMETER APPARATUS assembled for test with devices for cleansing shown adjacent.

poises) is expressed in the following manner:

$$\mu(\text{poises}) = \frac{[\text{Force}]\times[\text{Accel. Due to Gravity}]\times[\text{Length}]}{[\text{Area}]\times[\text{Velocity}]}$$

$$= \frac{[g]\times\left[\frac{\text{cm.}}{\text{sec.}^2}\right]\times[\text{cm.}]}{[\text{cm.}^2]\times\left[\frac{\text{cm.}}{\text{sec.}}\right]} = \frac{g}{\text{cm.}\times\text{sec.}}$$

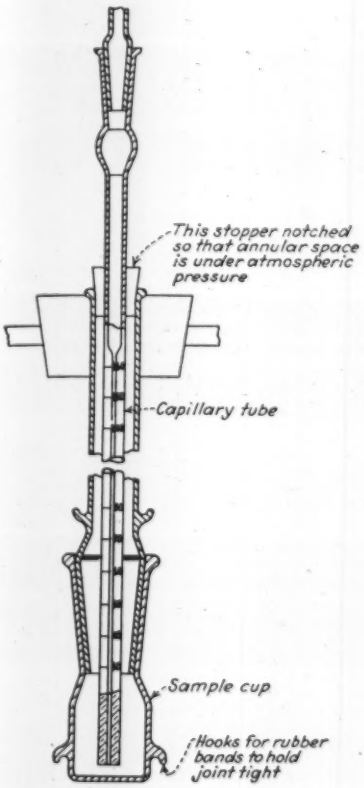

This stopper notched so that annular space is under atmospheric pressure
Capillary tube
Sample cup
Hooks for rubber bands to hold joint tight

FIG. 2—DETAIL of viscosimeter proper.

Since results determined by the viscosimeter are all expressed in poises and all grades of road tar can be tested at the same temperature, obviously no confusion can result and no conversion chart is needed when road tars of different consistencies are compared with this apparatus. On the other hand, it can be used, if desired, to determine the relationships between consistency values by usual test methods and absolute viscosities expressed in poises. Using the apparatus for this purpose, the road tar samples, previously tested by usual methods for the preparation of the conversion chart mentioned above, were tested for absolute viscosity at 35 and 50 deg. C. The results have been combined in a nomograph which permits the consistencies by the different empirical methods to be compared not only with each other but with absolute viscosities.

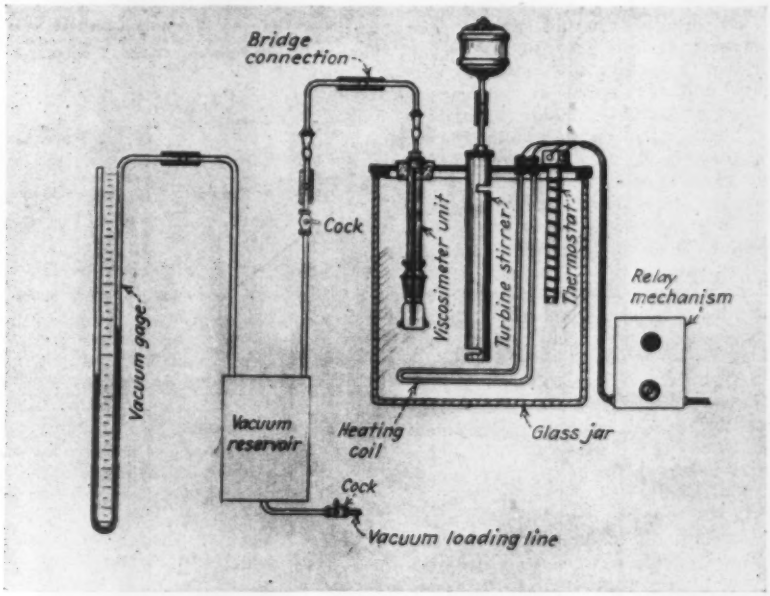


FIG. 3—SCHEMATIC ARRANGEMENT of apparatus shown by Fig. 1.

The table on p. 716, prepared from this nomograph, compares absolute viscosities, determined with the new instrument, and consistencies determined by some of the empirical tests commonly employed for the testing of road tars in the United States.

This discussion has been confined to the use of the Koppers Products Viscosimeter for determining road-tar consistencies because, as yet, other materials have not been tested with it. However, there is no apparent reason why the apparatus should not be equally satisfactory for testing all grades of road oils, road asphalts and many other kinds of fluid materials.

Description of viscosimeter

With this new instrument viscosity is determined by timing the flow of the material under test through a capillary tube. Contrary to the classical method of Ostwald, which utilizes gravitation as the driving force, the material under test is made to ascend through the capillary tube. This is accomplished by immersing in the sample to be tested the lower end of a capillary tube, the upper end of which is connected with a partly evacuated reservoir. Two fixed points are chosen on the capillary tube, and the time required for the passage of the air-sample interface between these points is noted. From this time, the vacuum applied, the distance between the fixed points and the radius of the bore of the capillary tube, the viscosity of the sample can be calculated.

The method described is not new. Pochettino ("Nuovo Cimento," Serie 6, 8 and 77, 1914) probably was the first to use it for the determination of the viscosity of bituminous materials. Recently it has been described again in connection with a study of coal tars at Columbia University. ("Dissertation," E. W. Volkmann, 1935.) However, no previous attempts have been made to develop a viscosimeter of this type suitable for rapid routine testing.

The instrument as constructed in the laboratories is illustrated by the accompanying view and drawings. Fig. 1 shows the general arrangement of the instrument and the auxiliary equipment used for cleaning the capillary tubes; Fig. 2 shows details of construction of the viscosimeter unit; and Fig. 3 shows a diagrammatic arrangement of the assembled instrument. The apparatus consists essentially of three parts.

The first part is the viscosimeter unit, which is shown in detail in Fig. 2. This unit is of all-glass construction with interchangeable parts. The most important part is the capillary tube, which must have a bore of uniform cross-section. Precision tubes with guaranteed dimensions can now be obtained from a few manufacturers. Ordinary capillary tubes may be used, but each tube must be calibrated carefully, since the diameter of the bore is an important factor in viscosity determinations.

The second part of the apparatus is the vacuum system, which consists of a vacuum reservoir, mercury U-gage, and necessary connecting lines. Fig. 3 shows these units diagrammatically. The vacuum reservoir is a brass tank of about 4-liter capacity. It is mounted inside the cabinet that supports the water bath (Fig. 1) and is provided with a loading line in the rear of the cabinet, which may be connected to any suitable vacuum supply. There are two other lines from the reservoir (Fig. 3). These connect to the mercury manometer (Figs. 1 and 3) and to the viscosimeter unit. The manometer is

made of heavy glass tubing. It is open to the reservoir at all times. The manometer scale is graduated in $\frac{1}{10}$ cm. The line from the reservoir to the Viscosity unit terminates in a standard tapered ground-glass joint. Connection to the viscosimeter unit is effected by a glass bridge that is removable. A gas-tight stopcock is provided in the above line below the ground-glass joint, at a location readily accessible to the operator. This cock is used to apply the vacuum and cut it off as desired.

The third part of the instrument is the water bath, which is used to obtain and maintain the desired temperature of the sample to be tested. This part consists of a glass jar of about 10-liter capacity. A brass coverplate is provided for the jar. On this coverplate are mounted the following units: (*a*) a turbine stirrer in the center of the plate, which is motor-driven as shown in Fig. 3; (*b*) a thermostatic control element and heating coil for maintaining bath temperature. The control element and heating coil are connected to a relay mechanism contained in a box mounted in the front of the cabinet (Fig. 1). This unit is capable of maintaining the bath temperature within limits of ±0.02 deg. C.

Holes are provided in the coverplate for the insertion of six viscosimeter units and a thermometer. Because of the central mounting of the stirrer, the whole coverplate may be rotated to bring any one of the viscosimeter units into position for test. In order to protect the glass jar from damage by rotation of the coverplate a spun brass ring is placed over the edge of the jar with rubber gaskets between the ring and the glass. Lugs soldered on the bottom of the coverplate fit inside this ring. Thus the coverplate is rotated on the brass ring, and the glass jar is not subjected to strain.

Besides the instrument itself, Fig. 1 shows the apparatus that was used for cleaning the viscosimeter units. This will be described under operation of the instrument.

CONSISTENCY RELATIONSHIPS FOR KOPPERS TARMAC

The samples tested in preparing this table were all made from one typical byproduct, coke-oven tar. Consistency relationships for road tars vary somewhat with variations in source. This table should not be used if accurate consistencies are required. It is intended to indicate approximate conversions for road tars in general.

Absolute Viscosity in Poises at 35° C.	Saybolt Furol, Seconds at 25° C.	Saybolt Furol, Seconds at 50° C.	Engler Specific, 50 cc. at 40° C.	Engler Specific, 50 cc. at 50° C.	Engler Specific, 50 cc. at 60° C.	Float Test, Seconds at 32° C.	Float Test, Seconds at 50° C.	Softening Points, Degrees C. by Cube in Water	Softening Points, Degrees C. by Ring & Ball	Present Standard Specifications for Std. Grades of Road Tar
20,000							220	46.7	38.2	
15,000							200	45.6	37.1	
12,500							188	44.9	36.3	TP-2
10,000							176	44.1	35.4	
8,000							165	43.2	34.5	
7,200							160	42.9	34.2	
6,500							155	42.4	33.7	
5,000							144	41.4	32.6	
4,000							134	40.5	31.7	TP-1
3,000							122	39.4	30.6	
2,500							116	38.7	29.8	
2,000						600	108	37.9	29.0	
1,500						420	100	36.8	27.8	
1,250						350	95	36.0	27.1	
1,000						298	90			
800						255	84			
650						224	79			
590						210	76			
500						191	72			TH-2
400						168	68			
320						150	63			
300						145	62			
250						134	59			
200						121	55			
150						109	50			
125						102	47			TH-1
100						95	44			
80		400			41.0	88	42			
65		352		90.4	36.0	82	39			
50		298		73.6	30.8	74	36			
40		259		62.5	27.0	67	33			
32		224		53.6	23.8	60	31			
30		214		51.4	23.0	59	30			
25		190		45.5	20.6	54	28			
20		164		39.0	18.0	47	26			
17.8		153		36.0	16.7	44	25			
15		134	83.6	32.0	15.0	40	23			TM-2
12.5		118	71.6	28.2	13.3	36	21			
11.2		109	64.0	26.0	12.4	33	20			
10.0		100	57.2	24.0	11.5	31	19			
8.9		92	51.0	22.0	10.6	28	18			
8.0		85	46.1	20.4	9.8	26	17			TM-1
6.5	890	72	38.2	17.4	8.5	22	15			
5.8	790	65	35.0	16.0	7.8	20	13			TC-4
5.0	660	59	30.2	14.2	7.0	18	12			
4.0	504	49	25.0	11.8	6.1					TC-3
3.0	364	39	19.1	9.3	4.9					
2.82	340	37	18.0	8.8	4.8					TC-2
2.50	294	34	16.0	7.9	4.4					
2.02	228	30	13.0	6.6	3.8					
2.00	225	29	12.9	6.5	3.8					TC-1
1.50	160	25	9.8	5.0	3.2					
1.25	137	24	8.0	4.2	2.8					
1.00	116	23	6.7	3.5	2.4					
0.80	102	22	5.6	3.1	2.2					

Operating procedure

The following test procedure is recommended for the determination of the absolute viscosities of road tars:

1. Establish the bath at a constant temperature. This temperature may be 35 deg. C., the lowest temperature that can be maintained during the warm summer months without providing cooling means for the bath. Higher temperatures may be used; however, the low temperature mentioned above resembles more closely average road conditions.

2. Place approximately 5 cc. of the road tar to be tested in the sample cup and assemble the viscosimeter unit. When assembled, the sample level in the cup should be ± 2 mm. from the 10 mm. mark on the capillary tube.

3. Immerse the viscosimeter unit in the bath. Twenty minutes should be allowed for the sample to come to bath temperature. This time is about double the time actually required for temperature equalization.

4. Evacuate the vacuum reservoir to the degree indicated by the apparent consistency of the material to be tested. Excess vacuum causes a too rapid rise of tar in the capillary, resulting in inaccurate timing. Too low vacuum prolongs the determination unnecessarily. With a little practice the selection of proper vacuum can be accomplished quickly and accurately.

5. Rotate the coverplate until a unit that has been in the bath for at least 20 min. is in position to be connected to the vacuum system. Then insert the bridge connection between the unit and the vacuum line.

6. Apply the vacuum by opening the cock in the line. The sample will now start to rise in the capillary tube. With a stopwatch take the time required for the interface to pass between two predetermined points on the capillary tube. For fluid materials the points are preferably: start at 40 mm., end at 120 mm. For more viscous materials the time interval between these two points would be very long; therefore, the following points are taken: start at 20 mm., end at 40 mm.

7. At the end of the timing period, shut off the vacuum, remove the bridge connection, remove the viscosimeter unit for cleaning, and rotate the coverplate until the next unit is in position for test.

8. The following data are taken for each determination:

a. Vacuum applied.

b. Limits of flow in capillary tube (20-40 or 40-120).

c. Time required for flow between limits chosen.

These data, together with the constants of the capillary tube, which have been determined by calibration, suffice for the calculation of the absolute viscosity in poises according to the formulas shown below.

9. Cleaning of the viscosimeter unit is accomplished readily by means of the apparatus shown in Fig. 1. The unit to the left of the viscosimeter in Fig. 1 is a bath of hot nitrobenzene. The viscosimeter unit is taken apart, and the capillary is immersed in this bath. After a few minutes the nitrobenzene will have penetrated up through the entire thread. The material may then be blown out. The film remaining on the side walls of the capillary is removed by the equipment at the extreme left of the photograph. This equipment consists of two

separatory funnels with rubber tubes connected to the bottoms below the stopcocks. One contains fresh nitrobenzene. The capillary is attached to the rubber tube, and nitrobenzene is allowed to run through until clear. Usually about 20 cc. are sufficient. Then acetone is run through the capillary from the other funnel to remove the nitrobenzene. Air is now pulled or blown through to remove the acetone, and the capillary is ready for use again. The sample cup may be cleaned with any of the regular solvent and soap cleaners by using heat and a test-tube brush. The sample cup should be flushed out finally with acetone and dried. The unit is then ready for re-assembly and use.

Evaluation of test results

Pochettino calculated the viscosity (poises) from data obtained by the capillary-rise method by use of the formula:

$$\mu = \frac{g \times H \times r^2 \times \Delta t}{4 \times (l_2^2 - l_1^2)} \quad (1.)$$

Where

g = acceleration due to gravity (cm./sec.²).
H = vacuum applied to capillary (cm. of H_2O).
r = radius of capillary bore (cm.).
Δt = time interval (sec.)
l_1 = length of capillary filled with tar at start of time interval (cm.).
l_2 = length of capillary filled with tar at end of time interval (cm.).

This formula is inaccurate, particularly for low values of H, because it does not take into account the effect of the static head of the fluid column on the value of the driving force H. Therefore, the following revised formula was developed:

$$\mu = \frac{g \times r^2 \times \rho \times \Delta t}{8\left\{\left(\frac{H}{\rho}+\lambda\right) \times l_n \frac{\left(\frac{H}{\rho}+\lambda\right)-l_1}{\left(\frac{H}{\rho}+\lambda\right)-l_2} - (l_2 - l_1)\right\}} \quad (2.)$$

Where symbols are as for Pochettino's formula plus

ρ = specific gravity of fluid at test temperature.
λ = length of capillary submerged in sample (1.0 ± 0.2 cm.).
ln = logarithm to the base e.

This formula is difficult to use when values of the logarithmic expression are extremely small. More satisfactory results may be obtained by using the following formula, which is obtained by expanding the logarithmic expression into a series.

$$\mu = \frac{g \times r^2 \times \rho \times \Delta t}{8\left\{\frac{l_2^2 - l_1^2}{2\left(\frac{H}{\rho}+\lambda\right)} + \frac{l_2^3 - l_1^3}{3\left(\frac{H}{\rho}+\lambda\right)^2} + \frac{l_2^4 - l_1^4}{4\left(\frac{H}{\rho}+\lambda\right)^3} + \frac{l_2^5 - l_1^5}{5\left(\frac{H}{\rho}+\lambda\right)^4} + \ldots\right\}} \quad (3.)$$

Both formulas (2) and (3) are cumbersome and much too involved for routine testing calculations. However, for any given class of material, approximations may be made which will yield results sufficiently accurate for commercial work.

For tar-road materials ρ can be taken as a constant. The effect of this factor is greater in the testing of the more fluid tars; therefore ρ has been taken as 1.15, which is an average of the gravity of the more fluid road tars tested.

If a 5-cc. sample of material is used, the depth of immersion of the capillary is very close to 1.0 cm. Therefore, λ may be taken as 1.0. Slight variations (±0.2 cm.) have no appreciable effect.

If, in addition to the above, 980 is substituted for g and as recommended under standard test procedure: $l_1 = 4$ and $l_2 = 12$ cm., the formula becomes:

$$\mu = \frac{147 \times r^2 \times \Delta t}{\frac{64}{\left(\frac{H}{1.15}+1.0\right)} + \frac{555}{\left(\frac{H}{1.15}+1.0\right)^2} + \frac{5120}{\left(\frac{H}{1.15}+1.0\right)^3} + \frac{49562}{\left(\frac{H}{1.15}+1.0\right)^4} + \ldots} \quad (4.)$$

or

$$\frac{\mu}{\Delta t \times r^2} = \frac{147}{\frac{64}{\left(\frac{H}{1.15}+1.0\right)} + \frac{555}{\left(\frac{H}{1.15}+1.0\right)^2} + \frac{5120}{\left(\frac{H}{1.15}+1.0\right)^3} + \frac{49562}{\left(\frac{H}{1.15}+1.0\right)^4} + \ldots} \quad (5.)$$

It can be shown that the expression $\frac{\mu}{\Delta t \times r^2}$ increases linearly with the increase of H. By the method of least squares, the expression on the right-hand side of Eq. (5.) was found to approximate closely the function $(25.909h - 17.1627)$. Therefore, the final equation becomes for road tars:

$$\mu \text{ (poises)} = (25.909 \times h - 17.163) \times r^2 \times \Delta t \quad (6.)$$

Where h = vacuum in cm. of mercury.
Δt = Time of flow in seconds.
r = radius of bore of capillary tube.
μ = absolute viscosity in poises.

Values of μ calculated by Eq. (6.) deviate from those calculated by the theoretical Eq. (2.) by negligible amounts.

For example, assuming Eq. (2.) to be exact when actual values of ρ are used:

h in cm. Hg	Per Cent Error by Eq. (6)
59.1	0.003
36.9	0.006
22.1	0.020
7.39	0.063
3.69	0.016

It will be seen from the above table that Eq. (6.) applies over the entire range of the instrument. However, when testing very viscous materials, such as hot-application road tars at low temperatures (35 deg. C. or lower), the time interval for flow between the limits 4 and 12 cm. is too long for routine tests. In such instances the use of Eq. (1.) is recommended, since the vacuum applied may be of such magnitude (above 40 cm.) that the error involved in Eq. (1.) can be neglected.

Data obtained with the instrument in its present form indicate a precision of approximately 1 per cent in ranges up to about 500 poises. As the viscosity increases the precision also increases. This indicated precision is considerably better than that of the instruments now in use for consistency measurements of the same materials.

The accuracy of the new instrument has been checked by comparisons with empirical instruments, such as Saybolt and Engler viscosimeters. These comparisons were made by using existing conversion relations between results obtained with such instruments and absolute or kinematic viscosities. In addition, some of the samples used to establish the range of the new instrument were tested with a Hoeppler viscosimeter.

This instrument also gives results in absolute units but, due to its construction, must be standardized with fluids of known viscosity. The instrument used was standardized by the Physikalische Technische Reichsanstalt, Berlin. The Hoeppler results agreed with those obtained with the new instrument in testing samples having viscosities up to 700 poises. Comparisons could not be made above 700 poises because the Hoeppler instrument is not suitable for road tars having viscosities above this limit. However, the comparisons with the Hoeppler instrument, together with comparisons using the empirical instruments, are felt to substantiate the assumptions made in deriving the formulas for use with the new instrument.

Sometimes in the evaluation of bituminous road materials it is desirable to know not only the viscosity at an average service temperature but also the temperature susceptibility (change of viscosity with change in temperature).

Many investigators have shown that for most road tars the relation between absolute viscosity and temperature is a straight line if the logarithm of temperature in degrees Centigrade is plotted against the logarithm of viscosity in poises. If such a plot is made for any given road tar, the slope of the straight line, which may be called the viscosity index, is a measure of temperature susceptibility. This viscosity index may be calculated from the formula

$$\text{Viscosity index} = \frac{\log\mu_1 - \log\mu_2}{\log t_2 - \log t_1}$$

where μ = viscosity in poises at t deg. C.

Since the viscosity-temperature relation is a straight line when plotted as above, any two temperatures with the corresponding values of μ may be taken. In the Koppers Products Laboratories viscosities at 35 deg. C. and 50 deg. C. are used for road-tar viscosity index determinations. Using these temperatures, the range of the new instrument is not exceeded.

Advantages summarized

1. Wide Range: All grades of road tar can be tested.

2. Absolute Units: Consistencies of all grades of road tar are expressed in the same units (poises).

3. Temperature of Test: All grades of road tar can be tested at a single temperature within the range of normal summer temperatures.

4. Temperature Susceptibility: By determining the consistencies at two different temperatures, temperature susceptibilities can be determined.

5. Small Sample: A 5-cc. sample is sufficient for each test.

6. Accurate Temperature Co[ntrol]: Due to the small sample employed and compactness of the viscosimeter unit, accurate temperature control, required for precision measurements, is maintained.

7. Speed: At least six viscosity determinations can be made in 1½ hours, using a single instrument with three or more viscosimeter units.

8. Precision: Measurements indicate a precision of 1 per cent in the lower ranges and a much higher precision in the higher ranges. This is better than the precision of standard instruments now employed for equivalent viscosity determinations.

9. Adaptability: Although the Koppers Products Viscosimeter has been used thus far for road tars only, it should be suitable for the testing of all grades of road asphalts and for many other types and classes of material.

Design Analysis of Roller Gates for Movable Dams

Method of computing loads and structural stresses in roller gates exemplified by an analysis of an 80-ft. gate for the Mississippi River navigation dam at Alton, Ill.

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THIS ARTICLE presents for the first time the methods of load and stress analysis followed by the army engineers in roller-gate design for movable dams. The specific example employed is an 80-ft. gate for the Mississippi River navigation dam at Alton, Ill. So that maximum use for analysis may be made of the limited space of a single article, the reader is referred for the development history in the United States of roller-gate movable dams to *Engineering Record,* Aug. 2 and Dec. 18, 1918, and *Engineering News-Record,* Sept. 21, 1933, and March 29, 1934. The essential structural and operating features of roller gates are indicated by Figs. 1 and 2.

The principles of design of roller gates are essentially based upon a thorough and complete analysis of the loading conditions and determination of reaction values and stress investigations for various positions of gate. In this way the critical position of the gate is determined for maximum condition of loading, and the proper design is selected.

In the case of the Alton gates a load analysis was made for the gate in the submerged position of 3 ft. below normal, and also for the closed position, gate raised 3, 6 and 9 ft. respectively.

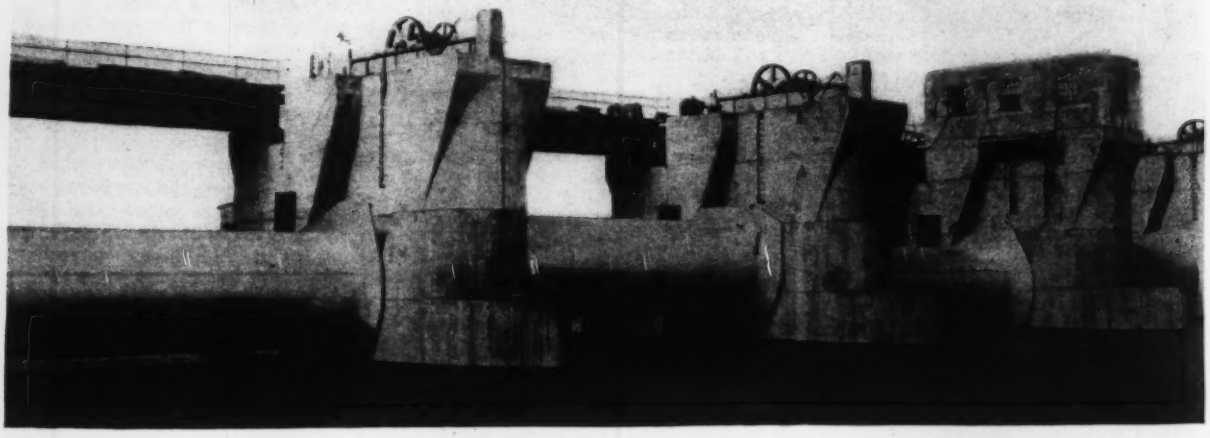


FIG. 1—TYPICAL roller-gate movable dam as constructed for navigation improvement of the upper Mississippi River.

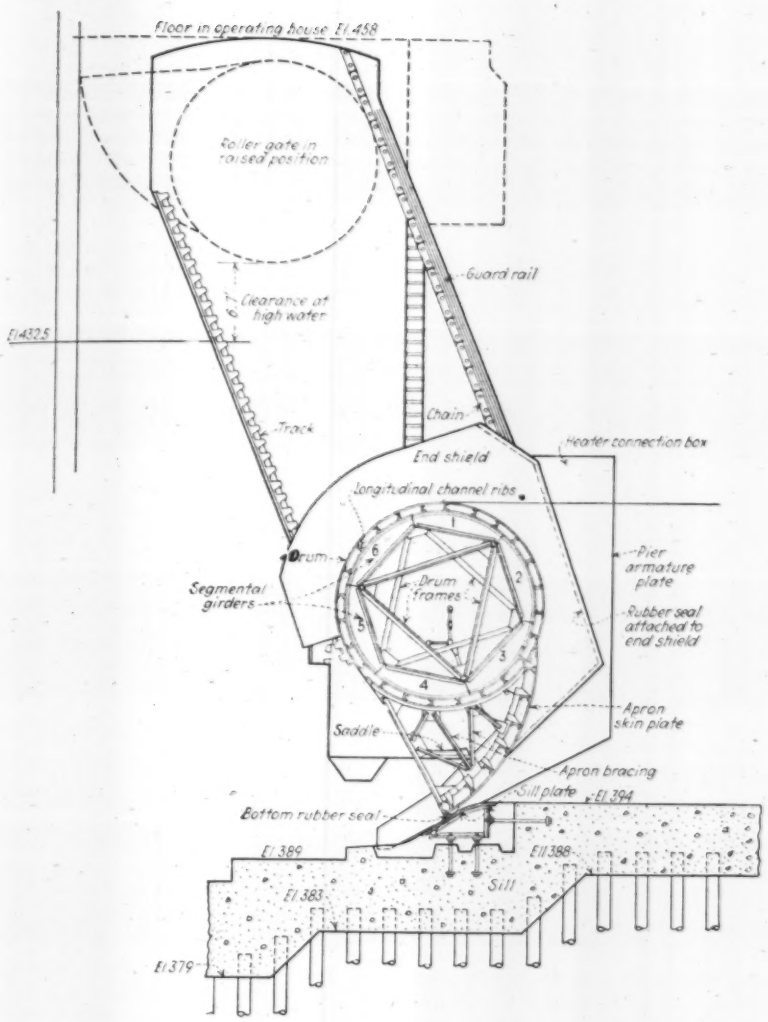


FIG. 2—SECTION OF ROLLER-GATE DAM at driven end, showing essential parts of roller-gate and track.

The loading condition that gave maximum stress is shown for the gate in the submerged position and was used in the analysis (Fig. 3).

Referring to this diagram, the water loads are drawn to scale as forces acting on drum and apron. The portion of the diagram that is shown cut off at the bottom is due to the reduction of water pressure between the apron and sill on account of orifice velocity at the restricted section. The theorem that all water loads act radially on a curved surface is shown graphically by means of radial lines drawn for the drum and apron respectively.

These forces are then drawn as vectors to some convenient scale for each linear-foot of curved surface, one resultant being obtained for the drum section and the other for the apron. The two resultants are combined vectorially and a new resultant is obtained. By graphical analysis this resultant was found to be 1,874,000 lb., which represents the total water load on the gate and was used in design. It will be noted in this connection that the magnitudes of the radial forces are equal to the horizontal forces projected to intersect the curved surface. This fact is derived from the theorem that *water pressure on any body under water at any point must be equal in all directions.*

Reaction analysis

The next step in the design was to make a reaction analysis of the gate in different positions.

The gate has five distinct points of support in all positions, except when resting on the saddle with the chain not acting. At the non-driven end the gate is supported by the track face and tooth of the gear segments, while at the driven end three other points of support are furnished by the track and tooth of the rack and rim and the chain in suspension. These five points of support establish the direction of all five reactions that pass through them, because of their construction and relation to each other.

A graphic solution is the simplest method of making a reaction analysis. Let R_W be the resultant water load on the gate in the submerged position, its magnitude and direction having been previously determined from the load analysis. The magnitude of this force is 1,874,000 lb., and when combined vectorially with W, the weight of the gate, assumed at 360,000 lb. and acting through the center of gravity of the gate section, a new resultant R is determined.

Reference is made to Fig. 4 for this analysis. The resultant R is then broken up into the five unknown reactions by the following method. The directions of these five reactions are all known, and the problem therefore resolves itself into determining their magnitudes or values.

First, extend the force R until it intersects the line of reaction of the chain and mark off the length R to scale. Divide the force R into equal vectors or reactions. The two forces, indicating T_n track load non-driven end, and Z_n tooth load, also non-driven end, are laid off normal to the track and tooth respectively, using the force $\frac{1}{5}$ R. The remaining three reactions at the driven end must then be determined. Based upon the fact that the track and tooth reactions must act through the point of contact of the rolling circle of the drum with the rack casting of the pier, a line is drawn through the point of contact from the intersection of the resultant R with the line of chain reaction. The vector C, or chain pull, is then laid off parallel to the direction of the chain as shown from the end of vector R to intersect the line drawn through the point of contact.

It is now possible to scale the vector C, or chain pull, which will be found to be equal to 305,000 lb.

To proceed from this point, the vectors Zd (tooth load) and Td (track load) are drawn normal as before to the tooth and track of the rack respectively at the driven end. Their magnitudes are determined from scaling as before, and a solution of the problem is therefore made.

The track load Td driven end equals 965,000 lb., and the track load Tn non-driven end equals 865,000 lb. The tooth load Zd driven end in this case equals 465,000 lb., and tooth load Zn non-driven end equals only 190,000 lb.

These values are all very essential in the design of the operating mechanism, racks and rims as well as in the design of the structural portions of the drum itself.

Stress analysis

The method of analysis for a practical design of the structural features of

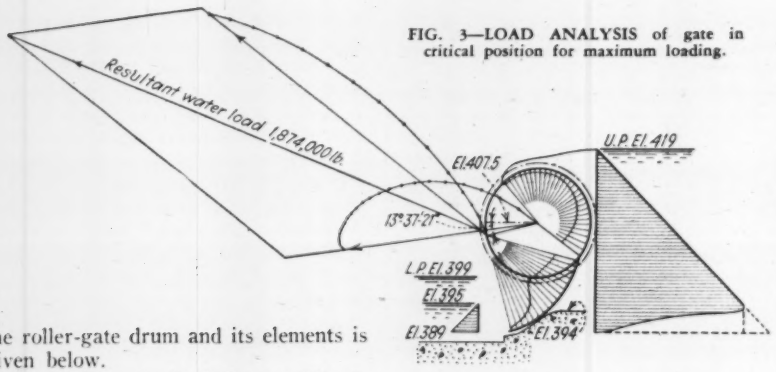


FIG. 3—LOAD ANALYSIS of gate in critical position for maximum loading.

the roller-gate drum and its elements is given below.

Referring to the load diagram Fig. 5, the uniform load R on the drum at the critical position equals $\frac{1,776,000}{80}$, or 22,000 lb. per lin.-ft. of drum where the loaded portion of the gate between piers is 80 ft.; but the span, center of contact points or c. to c. of load disks, is equal to 84 ft. 4½ in.

To obtain the reactions at the contact points of the drum, the chain load is included on the girder, and all loads are presumed to act in the same plane and directions as indicated. It will be noted that the chain pull for the gate submerged 3 ft., which is the critical position, is given as 305,000 lb. This load must be resolved with its component in the direction of the resultant R, in order to make use of it in the load diagram. The component in this case is found to be 165,000 lb.

The shear and moment diagrams are then plotted as in the ordinary manner for a girder loaded as shown. The computations that follow give maximum values only as used in design. For simplicity all loads and shears are given in kips or 1,000-lb. units and moments in foot-kips.

The weight of the gate is assumed as 1,000 lb. per foot uniformly loaded at the ends of the drum, acting in the same direction as the reactions, which is taken into account in the calculations.

The reactions and shears on the drum are shown on Fig. 5, as is the maximum bending moment (1,931.63 kips), which occurs at point 6 when the shear is zero. In addition, torsional moment (M_t) on the drum is produced by all the loads acting eccentric to its center. The increment of the torsional moment due to resultant load R per linear-foot of gate and with an eccentricity e of 5 ft. is $M_t = R \times e = 22.2 \times 5.0 = 111.0$ ft.-kips per foot of gate. Also the torsional moment at the non-driven end ($M_t n$) (Fig. 6) with a tooth load Z_n and eccentricity e to produce negative torsion is $-190 \times 9.33 = -1,773$ ft.-kips. Combining these two, the torsional moment at any point on the drum, at a distance x from the center line of load disk, non-driven end, is given by the formula:

$$M_t = -Z_n \times e + \Delta M_t(X - 2' - 2\tfrac{1}{2}'') = -1773 + 111 \times 40 = 2667 \text{ ft.-kips (for point 6).}$$

The maximum torsional moment at the driven end ($M_t d$) with a tooth load Zd and its eccentricity e combined with chain load C and its eccentricity e is given by the formula:

$$M_t d = Zd \times e + C \times e = 467 \times 9.33 + 305 \times 9.0 = 7104 \text{ ft.-kips.}$$

For check, the torsional moment at the driven end may also be figured by using the following formula, where L equals the loaded span.

$$M_t d = -M_t n \times \Delta M_t \times L = -1773 + 111 \times 80 = 7107 \text{ ft.-kips.}$$

The formula for combined moments M on any section of the drum as derived from mechanics is:

$$M = \tfrac{1}{2}(M_b + \sqrt{M_b^2 + M_t^2})$$

For maximum moment at point 6

$$M = \tfrac{1}{2}(19,543 + \sqrt{19,543^2 + 2667^2}) = 19,636 \text{ ft.-kips (Fig. 5).}$$

For all other values of moments along the drum reference may be made to the roller-gate moment table given in Fig. 6. These values were all obtained by making use of the calculating machine for checking purposes. However, the slide-rule calculations are sufficiently accurate for design and can be used in this type of work.

From this point in the calculations, design of the drum proper may proceed or an analysis of stresses can be made from the design.

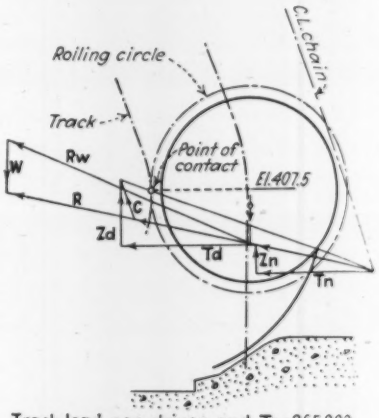
The moment of inertia of the drum plate is $I = \pi R^3 t$ where R is radius to center line of drum plate and t is the thickness of the plate. For the combined moment of inertia I, including the longitudinal channels, the formula may be expressed thus:

$I = \pi R^3 t + \Sigma I_o + \Sigma A d^2$, which gives for a $\frac{5}{8}$-in. plate with radius of 101.81 in. and twenty-four 7-in. 17.6-lb. ship channels whose distance d from the center of the drum is 98 in., a moment of inertia I of 2,345,124 in.

The section modulus of the drum section is $S = \frac{I}{R + t/2} = \frac{2,345,124}{102.125} = 22,963$ in., so that the maximum stress (at W section 6) is $f = \frac{M}{S} = \frac{19,636 \times 12}{22,963} = 10,261$ lb. per sq.in.

The allowable stress in bending for the drum, exclusive of the apron, is 16,000 lb. per sq.in. for tension or compression. Apron action was neglected in the design but may add considerable strength to the drum section, particularly on the compression side. The maximum fiber stress in tension was figured on the net section of the drum, assuming the neutral axis on the center line of the drum. This is an approximate solution but on the safe side.

The stresses in the drum section were intentionally kept low, in order to allow for additional stresses due to temperature changes and friction at contact



Track load, non-driven end	Tn = 865,000
Tooth load, non-driven end	Zn = 190,000
Track load, driven end	Td = 965,000
Tooth load, driven end	Zd = 465,000
Chain reaction	C = 305,000
Resultant water	Rw = 1874,000
	R = 1776,000

FIG. 4—REACTION ANALYSIS of gate in critical position for maximum loading.

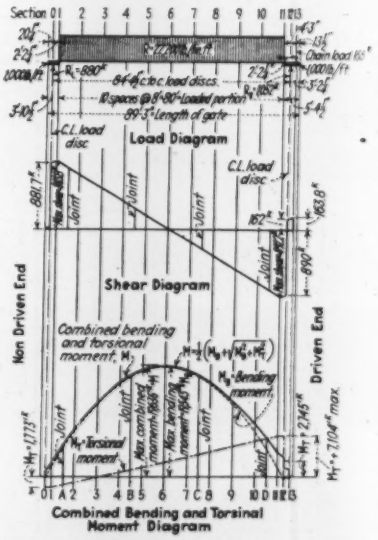


FIG. 5—LOAD, SHEAR and combined bending and torsional moment diagrams for roller gate.

points during operation. These factors may add as much as 10 per cent to the total stresses in bending, due to direct load and torsion.

Curves for unit shear and bending stresses, shown in Fig. 6, were plotted for critical points along the drum. Particular attention was given to the calculation of the stresses at the circumferential splices. It will be noted that the stresses are higher at these points due to the holes required for riveting. All splices were designed for both moment and shear, and sufficient rivets were provided to take the stresses determined from the diagrams.

The efficiency of the circumferential splices is determined by computing the ratio of the net section to the gross section at the first line of rivets of the splice-plate connection to drum plate. Using this efficiency and computing, in turn, the net moment of inertia, the section modulus and bending moment at a given joint, the stress is determined.

At joint *B* there are 168 round holes (1⅜-in. diameter), which were deducted from the ⅝-in. drum plate, giving a splice efficiency equal to 77 per cent. The bending-stress diagram (Fig. 6) shows the effect of the splice efficiency for the actual stress at all joints on the drum.

Shearing stresses in the drum plate are produced by a combination of direct loading and torsion, due to eccentric reactions and loads.

For points *X* and *Y* (Fig. 6) the shearing stresses at the ends of the drum are: Direct shear $v_1 = \frac{V}{\pi Rt}$ and torsional shear $v_2 = \frac{2M_t}{\pi d^2 t}$.

For the driven end of the drum, assuming net area of plate as 80 per cent of gross area,

$$v_1 = \frac{892{,}200}{\pi \times 101.75 \times 0.5 \times 0.80} = 6{,}977 \text{ lb. per sq.in.}$$

$$v_2 = \frac{2 \times 7{,}104{,}000 \times 12}{\pi \times 203.5^2 \times 0.5 \times .80} = 3{,}276 \text{ lb. per sq.in.}$$

The total shear ($v = v_1 + v_2$) at the driven end of the drum for point *Y* is 6,977 + 3,276 = 10,253 lb. per sq.in., and for point *X* is 6,977 — 3,276 = 3,701 lb. per sq.in. For point *X* torsional shear is deducted from direct shear because torsion acts in the opposite direction to the direct stress in shear.

Shearing stresses at the non-driven end of the drum are somewhat less, as shown in Fig. 6. Maximum shearing stress was found to be at joint *D*, where the splice was 72 per cent efficient.

The value for shear at point *Y*, driven end of drum, exceeds the allowable of 10,000 lb. per sq.in. by less than 4 per cent, which was considered satisfactory.

In addition to direct compressive stresses in the drum section at the extreme fiber as computed above, an additional stress due to local bending of the channels is introduced and must be considered in design. This stress must be added as a compressive stress at the extreme fiber of the drum section. In calculating this stress the assumption was made that one-half the load is carried by the drum in arch action and one-half is effective in producing bending stress. This assumption was found to be conservative from subsequent tests made on actual roller gates under loading. It will be noted that the longitudinal framing of the drum consists of twenty-four 7-in. 17.6-lb. ship channels spaced at 2 ft. 2⅛ in. c. to c., or at intervals of 15 deg. with the center of the drum and supported by means of drum bracing frames spaced 8 ft. c. to c.

Consequently the maximum bending moment *M* of the channel ribs is figured as in a continuous beam.

$M = \pm 1/12\ WL$ (positive and negative moments being equal).

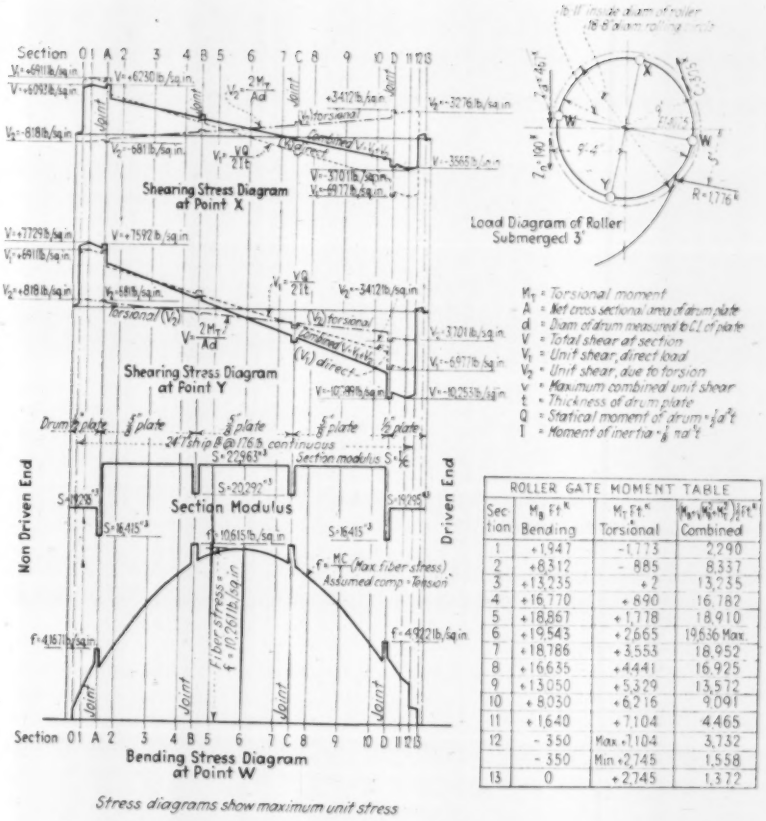
Total load = $W = 2.22 \times 14$ ft. 8 in. $\times 62.5 \times 8.0 = 16{,}286$ lb.,

and $M = \frac{16{,}286 \times 8}{12} = 10.86$ ft.-kips.

Assuming that a 12-in. width of ⅝-in. skin plate acts with the channel rib as a tee-beam, the center of gravity, moment of inertia, section modulus and maximum stress of this beam are computed in turn, giving a maximum stress in local compression for the channels of 2,980 lb. per sq.in. Therefore, the total compressive stress in the drum section, including local bending, is $f_c = 10{,}260 + \frac{2{,}980}{2} = 11{,}750$ lb., whereas the allowable stress is 13,500 lb. per sq.in.

Tests have shown that the apron plate and framing, which are necessarily essential parts of the roller gate, will materially reduce this stress, but conservative practice in the past was to neglect this effect. On the other hand, unavoidable factors in the operation of the gate will tend to increase the stresses as the gate moves into and from the critical position. This is particularly true during the winter months when the gate is forced to break the ice sheet of the upper pool, and also in the summer months due to expansion of the steel plate facing the sun's rays. Because of these factors, it was not considered advisable to exceed 16,000 lb. per sq. in. in tension and 13,500 lb. per sq. in. in compression, for the drum, acting only under direct water loading but neglecting the effect of the apron entirely.

The details of the drum proper include twenty 7-in. 17.6-lb. ship channels and four 7-in. 25-lb. ship channels for


Shearing Stress Diagram at Point X
Shearing Stress Diagram at Point Y
Load Diagram of Roller Submerged 3'
Section Modulus
Bending Stress Diagram at Point W
Non Driven End
Driven End
Joint
Stress diagrams show maximum unit stress

M_T = Torsional moment
A = Net cross sectional area of drum plate
d = Diam of drum measured to C.L. of plate
V = Total shear at section
V_1 = Unit shear, direct load
V_2 = Unit shear, due to torsion
v = Maximum combined unit shear
t = Thickness of drum plate
Q = Statical moment of drum = $\frac{1}{2}a^2t$
I = Moment of inertia = $\frac{1}{8}\pi a^3 t$

ROLLER GATE MOMENT TABLE

Section	M_B Ft.K Bending	M_T Ft.K Torsional	$(M_B v_1 + M_T v_2)\frac{1}{2}$ Ft.K Combined
1	+1,947	-1,773	2,290
2	+8,312	- 885	8,337
3	+13,235	+ 2	13,235
4	+16,770	+ 890	16,782
5	+18,867	+ 1,778	18,910
6	+19,543	+ 2665	19,636 Max.
7	+18,786	+ 3,553	18,952
8	+16,635	+4441	16,925
9	+13,050	+5,329	13,572
10	+ 8,030	+6,216	9,091
11	+ 1,640	+7,104	4,465
12	- 350	Max +7,104	3,732
	- 350	Min +2,745	1,558
13	0	+2,745	1,372

FIG. 6—LOAD, shearing-stress and section-modulus diagrams for roller gate.

longitudinal ribs and a $\frac{5}{8}$-in. skin-plate covering for the major position of the gate. The two ends, however, have $\frac{1}{2}$-in. skin plates. All stresses as analyzed above indicate safe and conservative design.

The longitudinal channels are supported at 8-ft. intervals by means of drum frames consisting of two triangular frames braced against the ends of a series of six segmental girders. Stresses have been analyzed for these drum frames, but due to statically indeterminate conditions at the joints two assumptions were made in order to arrive at a reasonable solution.

The first assumption was that all compressive stresses are taken by the triangular frames, each frame independent of the other and the joints hinged. The segmental girders in this case were assumed to take the bending stresses only and their reactions, figured to go directly into the triangular frames in compression. An analysis of this case gave a maximum compressive stress in one of the diagonals amounting to 20,800 lb. The members selected for this stress were 5x5x$\frac{7}{8}$-in. angles, whose area of 4.18 sq. in. gave a unit stress of 4,980 lb. per sq. in.

The allowable stress in these angles for compression $\left(16{,}000 - 70\frac{1}{r}\right)$ is 6,550 lb. per sq. in.

The second assumption, that the segmental girders take the water loads in bending and direct compression at the points, determined the design of these members. The segmental girders are made up of $\frac{3}{8}$-in. web plate and single 3x3x$\frac{3}{8}$-in. flange angles. The combination of these stresses is well below the allowable.

The apron framing was designed for local bending only. The longitudinal channels are supported on apron frames spaced 8 ft. c. to c. as for the drum. In this case the drum acts as a supporting girder for all water loads applied on the apron skin plate. There are eight longitudinal 10-in. 20-lb. ship channels and a $\frac{1}{2}$-in. skin plate, including bracing, etc., for the apron design.

Load Disk—A short analysis is given below for the design of the load disk at the driven end of the gate. The track load T_d at this point is equal to 965,000 lb. Assuming that the load will spread 45 deg., the pressure can be taken on a width B of 20 in. at the periphery of the disk. Therefore, the required thickness of metal for a bearing value f_b of 24,000 lb. per sq.in. is

$$t = \frac{T_d}{B \times f_b} = \frac{965{,}000}{20 \times 24{,}000} = 2.01 \text{ in.}$$

The design of the load disk specifies a solid $\frac{3}{4}$-in. disk plate reinforced with a $\frac{3}{4}$-in. segmental and an 8x8x$\frac{3}{4}$-in. flange angle on the outside edge of the disk. This material is necessarily concentrated at the point of maximum track loading, but a reduction of metal is possible at other points on the disk as the load distribution is reduced toward the center. For all other positions of the gate, as it is raised on the track, a consequent reduction of the metal can be made.

Sufficient and proper stiffener angles and channels milled for bearing are provided for bracing the plate against buckling under the enormous track loads. Although the track load T_n for the non-drivers end is equal to 865,000 lb., which is less than the load at the driven end, it was deemed advisable to make both load disks alike.

End Disks—Although the end disks of the drum do not carry any direct loading from the rack or rim either from a tooth or track reaction, some stresses are, however, transferred to the disk at the driven end by means of the chain load during the operation of the gate. Due to this fact a substantial framing is recommended for the end disks.

Chain Segment—The short segmental girder or chain segment to which the chain hook is attached is located between the load disk and end disk at the driven end of the gate. The maximum chain pull is equal to 305,000 lb. and acts tangentially to the drum.

The chain hook is made up of two steel plates, $2\frac{3}{8}$ in. thick on the outside of the drum, and machined to 1-in. thickness for the inside. Both plates are riveted to the $\frac{3}{4}$-in. web of the segmental girders and welded on the outside to the skin plate of the drum.

There are about ninety $\frac{7}{8}$-in. rivets connecting the plates of the chain hook to the web of the segmental girder and 120 lin-in. of $\frac{3}{8}$-in. fillet weld between the chain hook and the drum.

The resistance of both the rivets and weld can be considered in taking the torsional and shearing stresses due to chain pull.

End Shield—A very important feature of the roller gate is the proper size and shape of the end shields, which are attached to the drum and apron and adjacent to the armature plate of the piers. In order to secure an effective seal between the end shields and armature plate, a rubber seal is attached to the end of the shield. The end shield has a $\frac{7}{8}$-in. plate, with some bracing and brackets but not sufficient for a rigid structure, yet of sufficient strength to take some unbalanced water pressure against one face. Flexibility in the operation and good contact for the seal are the desired factors in this instance, and experience has shown that this type of shield has been successful on the former gates designed by the U. S. Engineers, and it was used for the roller gates of the Alton Dam.

The author is grateful to the following engineers in the St. Louis division engineer office for furnishing material and assisting in the preparation of this article: V. A. Vollmer, J. C. Harold and H. E. Boath. The design of the Alton locks and dam was prepared under the direction of Col. E. L. Daley. Lt. Col. P. S. Reinecke is in charge of construction. John Griffiths and Son Co., of Chicago, have the contract for the locks, and the Engineering Construction Corp., of New York, for the dam.

Letters to the Editor

Concrete Beam and Cylinder Strength

Sir—The article by C. B. McCullough, "Flexural Resistance of Shallow Concrete Beams" (*ENR*, Sept. 19, 1935, p. 406) and the discussion by H. M. Hadley (*ENR*, Oct. 31, 1935, p. 617) may serve to focus attention on the timely subject of working stresses in reinforced-concrete beams. The results of these tests are not extraordinary as might be inferred from the article, but they confirm very forcibly the conclusions announced by Slater and Zipprodt (*Proc.*, American Concrete, 1920, p. 120-146) and Slater and Lyse (*Proc.*, A.C.I., 1930, p. 831-874). These earlier studies, based upon a large number of beam tests, showed that the ultimate compressive stress in a reinforced beam (computed according to the usual straight-line theory and using the modular ratio, n, as given by the A.C.I. Building Code) varied from about two times the cylinder strength for 1,500-lb. concrete to about 1.5 times the cylinder strength for 3,000 to 6,000-lb. concrete. The author's values, averaging about 1.74 times the cylinder strength of 3,950 lb. per sq. in. for beams which actually failed in compression, are not greatly in excess of what would be expected from knowledge of earlier tests.

Working stresses in flexural compression permitted by the 1924 Joint Committee Report and the 1928 A.C.I. Standard Building Code are $0.4f'_c$ in general, and $0.45f'_c$ near supports of continuous beams and frames (where f'_c is the 28-day cylinder strength). Considering an ultimate fiber stress of $1.5f'_c$ as representing a minimum value for all grades of concrete, it is seen that the factors of safety are at least 3.75 and 3.33, respectively, for the two cases. The author's comparison of his test values with a working stress of 650 lb. per sq. in. may be intended as *reductio ad absurdum:* few enlightened engineers would use 650 lb. per sq. in. on 4,000-lb. concrete when the Joint Committee Report advocates the use of 1,600 to 1,800 lb. per sq. in.

The factor of safety of 3.33 or more in compression may be contrasted with

a factor of safety of 2.0 to 2.5 in the tension reinforcement, assuming that working stresses of 16,000 lb. sq. in. for structural steel or 20,000 lb. per sq. in. for intermediate grade are used. It is evident that in the usual current design the tensile steel governs the ultimate strength of a beam or slab. The factor of safety against diagonal tension failure is ample when reinforcement is provided in accordance with the usual stirrup and bent-bar formulas, but the factor of safety against bond failure is not so satisfactory. It is not difficult to design a beam meeting the requirements of many codes, which will show a factor of safety in bond of less than 2.0. Evidently the working stresses for the various elements of beam strength might well be revised if we are to have balanced design of the "one hoss shay" type. A practical deterrent to this condition has been the reluctance of many engineers to use high concrete stresses because of the lack of inspection necessary to insure delivery of concrete of the quality specified. Also, as indicated by the author, there are usually economic limitations on the use of the high percentages of reinforcement required to develop the potentialities of high-strength concrete.

F. E. RICHART,
Research Professor of Engineering Materials, University of Illinois.

Urbana, Ill., Nov. 5, 1935.

Flexural Resistance of Shallow Concrete Beams

Sir—Referring to the report of Conde B. McCullough on tests made on shallow concrete beams (*ENR*, Sept. 9, 1935, p. 406), it is desired to call attention to additional information derived by these tests.

Where the beams failed by rupture of the concrete at the top, or in flexure (denoted "Compression Failure, Concrete" in the table) a maximum unit stress greatly in excess of the ultimate compressive stress in the cylinders was computed by Mr. McCullough. It is not believed that the stress in flexure could have been appreciably more than the test-cylinder results.

The method of computing the stress in the top of a reinforced-concrete beam during the final loads should be different from that used for loads which produce proportional distortion of the steel and concrete. In the study of tests of this type, the writer has used a formula developed from the stress prism of the concrete above the reinforcing steel. In this formula F_c is equal to the maximum unit stress in the top of the beam, b is the width of the beam, d is the effective depth, and the center of gravity of the stress prism above the reinforcing steel would be $\frac{3}{4}d$. The internal resisting moment may be expressed as:

$$M = (F_c b \frac{d}{2}) \frac{3}{4}d = \frac{3}{8} F_c b d^2$$

In beam type 3 of Mr. McCullough's table, having 8.94 sq.in. of reinforcing steel, the concrete failed in flexure, and the unit stress in the concrete is given as 6,938 lb.; the ultimate load was 118,800 lb. The external bending moment was approximately 362,500 lb., and the maximum unit stress when computed by the formula given above would be 4,550 lb. per sq.in., compared with the given cylinder strength of 4,342 lb. The maximum unit stress in the other concrete beams where the concrete failed in flexure may be computed in the same way, giving results more nearly in keeping with the cylinder results.

The information derived in the tests of these beams might be put in a form that would be of assistance to the structural engineer in the design of concrete beams. The material used was installed by field methods and was for all practicable purposes 4,000-lb. concrete. The sections were rectangular and with a depth equal to about 1 in. per foot of span. If the internal resisting moment is represented by Kbd^2, where b is 24 in., d is 10 in. and K is a constant depending on the reinforcing steel, grade of concrete and other factors, a table similar to the partial one following may be made up.

Per Cent of Reinf. Steel	K Type 1 Beam	K Type 2 Beam	K Type 1 Safe Load	K Type 2 Safe Load
0.0102	575	534	190	178
0.0200	1048	1073	349	360
0.0295	1043	1204	345	401
0.0373	1160	1319	386	439
0.0460	1230	1490	402	496

The values of K given in the "Safe Load" columns have been obtained by dividing that for the breaking load by 3. This is a conservative factor of safety, although the value of K obtained is much larger than would be usual in standard requirements.

G. L. BILDERBECK.
Groton, Conn.

Oct. 24, 1935.

Flat Shell Domes of Reinforced Concrete

Sir—We have read with much interest the article in your issue of July 25, 1935, describing the shell dome of the Hayden Planetarium, by Paul F. Pape, your editorial in the same issue and the letter of J. W. Pearl, of Chicago, describing the shell dome at Hammond, Ind., in your October 31, 1935, issue.

Your editorial refers to the shell dome as "extremely rare in the United States." We believe the flat shell dome of reinforced concrete has been used more extensively in the United Stattes than in any other country. We have designed numerous structures of this character. We have several roof domes of 75-ft. span, $\frac{1}{8}$ rise and 3 in. thick throughout. One of these domes at El Centro, Calif., has been subjected to numerous earthquake shocks without showing any signs of distress. We have several domes 100 ft. and more, and one of 140-ft. span, 4 in. thick at the crown. We are now designing a 160-ft. span. Our designs all call for a $\frac{1}{8}$ rise unless a greater rise is required for architectural reasons.

The hemispherical dome is a very simple structure both in design and execution, needs very little reinforcing and no special treatment. The flat dome requires an unyielding abutment to prevent excessive bending. All domes are subjected to bending. It is true that ring stresses resist bending, but it is also true that if there were no bending, there would be no ring stress.

Where a flat shell dome is furnished with an abutment heavy enough to be practically unyielding as in the Hammond, Ind., dome, or when the unyielding quality is given the abutment by prestressing the steel bands of the abutment, then the bending in the dome slab is very slight and may be neglected. If the abutment of the structure yields materially due to the elasticity of its steel reinforcing, the result is a bending moment in the slab of material magnitude, which is difficult if not impossible of accurate solution and results in a thoroughly dangerous structure.

We believe a flat shell dome with a rise of $\frac{1}{10}$ to $\frac{1}{8}$ of the span is the most economical form of roof possible for circular structures. It could be employed to great advantage for buildings requiring clear floor space. A flat shell dome of reniforced concrete costs from 60c. to $1 per sq.ft. of horizontal area. We know of no other form of fireproof roof of similar span that would not exceed this cost. A larger rise than $\frac{1}{8}$ of the span requires back forms and is not economical.

Domes may be economically built for large loads. We have a dome at Findlay, Ohio, with a diameter of 80 ft., 8-ft. rise, which carries a total load of 1,200 tons, much of it concentrated at the center. We have numerous domes of similar capacity. We have several domes carrying a water load of more than 1,000 lb. per sq.ft. Domes of 250- to 300-ft. span are both practical and economical.

The forms for casting the dome slab are very simple in construction when properly designed. For a hemispherical dome, radial ribs should be used, but for a flat slab dome with a rise of $\frac{1}{8}$ to $\frac{1}{4}$ of the diameter, parallel ribs are more economical.

A dome of 80-ft. span, 11-ft. rise and 1½-in. thickness was built in California several years ago and is still standing. Such a structure, however, could hardly be regarded as safe. A slight yielding of the abutment or a concentrated load would surely result in failure.

The economical and architectural features of a thin slab dome justify much greater consideration from architects and engineers.

WILLIAM S. HEWETT,
William S. Hewett System.
Chicago, Ill.

November 4, 1935.

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November 21, 1935

A Promising Revival

FROM VARIOUS SECTIONS of the country comes news of urgent demand for engineers, in several instances exceeding the supply and creating a possible shortage of technical personnel. Something the same is reported as to architectural designers and draftsmen in several centers. These reports are evidence that planning is once more entering an active stage, in preparation for the increased construction that is just ahead and that, in fact, has already begun in considerable measure. Plant expansion, housing construction and public works are each drawing on the supply of engineering personnel, and while in some fields the heavy demand may prove to be shortlived there is good promise in the existing situation that the sum total of engineering activity will increase steadily. For the first time in some years, thus, engineering employment prospects are becoming favorable. And, since planning is the forerunner of construction, the outlook for construction activity also is made brighter by the current reports.

Unfavorable Dam Sites

FRENCH ENGINEERS took a bold step forward when they built the Sautet Dam. Height alone would make the structure notable, since it is exceeded only by Boulder Dam; but it is even more notable for its compound structure—the combination of a thin constant-angle arch with a backing of lean concrete that thickens the structure to nearly gravity section, though its chief function is to buttress the canyon walls, check erosion and prevent rock falls, which otherwise would threaten the abutments of the thin arch. Through this combination a saving both in concrete and in rock excavation was realized. Unusually extensive grouting of the limestone walls was carried out, and galleries have been left open to observe changes that may take place as the reservoir fills. These features of the structure are due to the nature of the site, which in the character of its rock is by no means ideal for a high dam. Sautet thus exhibits problems that will be placed before engineers in increasing number as the more favorable dam sites are taken up and naturally unfavorable sites must be converted into safe sites at reasonable cost.

New Results in Grouting

IF GROUTING WAS OF DECISIVE IMPORTANCE in connection with Sautet, its use at the Norris Dam has even broader significance because of the new results accomplished by the process there. The replacement of layers of soft material between the foundation rock strata by a solid cement filling gives an assurance of support to the dam that varying foundation conditions often make essential. Previous success on other jobs in cleaning out seams and fissures in preparation for grouting does not take away from the Norris work any of the credit of original procedure. Adding to the excellence of the accomplishment is the large-size core-drill probing of the results, which with "periscope" inspection through smaller drill holes provided a gratifying verification of the adequacy of the work. Foundation and abutment grouting in all the major dams for the last few years has been of such a character as to build up a new art of high value to future construction. The work at Norris further extends the possibilities of this new art.

Laboratory Contributions

THE EXTENT to which mechanical invention takes root in the research laboratory is not often fully appreciated. A case particularly in point is the new viscosimeter described in this issue, but quick recollection calls to mind the Harvard Graduate School apparatus for multiple soil tests, the proving-ring device for measuring heavy earth pressures on walls, the new water-pressure device of Norwegian research workers to determine the clogging value of different mixes for concrete aqueducts, and many other valuable engineering aids that came from the laboratory in the last two years. With few exceptions these devices are the invention of the research worker, who called upon the instrument maker only for his precision and craftsmanship. It is quite natural that the source of invention should be where it is; there is small commercial market for any of these devices, and the nature of physical research is such that often the problem to be met is not known until revealed in the course of research. But the proficiency with which the laboratory worker is constantly stepping outside the specific field of his work to the production of his working tools deserves praise that engineers often forget that they owe.

Efficient Structures

MARKED PROGRESS has been apparent during the past half-dozen years or more in placing structural design, more thoroughly on a basis of sound scientific understanding, and casting away certain outworn traditions. Prominent among the latter was the doctrine that continuous bridge construction is questionable, because of uncertainty of analysis, possible settlement of supports, and other imaginary hazards. When this ancient belief was discarded continuous bridges came to recognition, first in long-span railway bridge construction (e.g. Scioto-ville) and some years later in the short-span highway field. The latter especially has proved to offer many fitting opportunities for continuous construction, and in each case economy and a more efficient structure were the result. The extensive use of deep, wide-flange rolled I-sections, in whose application Georgia was a leader on the initiative of S. B. Slack, gave impetus to this development. The consistent application of continuous construction by the Kansas department, as described by George W. Lamb in this issue, offers an effective illustration of how useful this modernization of construction type can be made.

What Can Be Done

WHEN THE REPRESENTATIVES of industry go to Washington early next month to discuss the question of a new NRA system of regulation, in the conferences called by Coordinator George L. Berry, they will be concerned with two questions: first, the question of establishing and safeguarding sound business practices; second, the question of improving the position of workers. These

questions have nothing in common, and the biggest contribution that industry can make in the December conferences is to keep this fact in the forefront of attention and insist on their complete separation. It is notorious that the attempt to merge them was a prime cause of the downfall of the original NRA; for, while the Supreme Court decision in the Schechter case turned largely on a trade practice, the real issue was the attempt at arbitrary centralized control which grew out of the joining of labor protection with fair-practice regulation. There is need for raising the level of quality and fairness in industry, and since experience shows that automatic processes are not always sufficient for the purpose it will be necessary to supplement these by supervision in the public interest. Similarly there is need for stabilizing employee relations by protecting the worker against the employer and the employer against the worker; here also public supervision like that now exercised over safety and other phases of worker relations is essential. But these two fields are fundamentally different in every respect. If the Washington conferences insist on merging them in the same discussion, it is almost certain that nothing will result except fresh discord.

In a New Stage

THE federal government's effort to make the Electro-Metallurgical Co. take out a Federal Power Commission license for its Hawk's Nest hydro plant on the New River has entered a new stage. So far the government has been thwarted in all its more or less direct moves to have the power company admit, or the courts declare, that this plant, located on the non-navigable New River in West Virginia, might so change the regimen of the New River as to affect the navigability of the Kanawha River into which it flows, thereby bringing the power plant within federal jurisdiction. The state of West Virginia has aided the power company in its fight, which has attained national significance; if the federal government were to win its point on the issue as heretofore drawn, its jurisdiction would automatically be extended from the navigable sections of rivers to the full extent of the stream system.

The most recent court skirmish ended inconclusively when the Supreme Court dismissed the case then before it because it found no cause of controversy between the federal government and the state of West Virginia. At that time the government's lawyers made an obscure reference to the fact that the federal government was purchasing land at the headwaters of the New River. A few days ago the true significance of that statement came out when it was announced that the army engineers had been allotted funds for preliminary work on Bluestone Reservoir, in the headwaters of the New River, a reservoir to be built for flood and erosion control and for power development.

By going above the Hawk's Nest plant to store water which could be used to improve the navigability of the Kanawha, the government hopes to perfect its case against the power company. We are not lawyers, but we think that it can be demonstrated to the satisfaction of engineers at least that the Hawk's Nest plant still could be so operated as not to interfere in any material way with the flow from the storage reservoir above. But the government's lawyers may be able to get the Supreme Court to take the opposite view. If they do, they will have won the now-famous New River case, but its significance will have departed with the start of work on the Bluestone Reservoir. The fundamental question raised originally will no longer be before the court, for construction of a government reservoir above Hawk's Nest will have converted it to a special case.

Sound Principle

BY EMPHASIZING the fact that the prime function of present-day federal land reclamation is salvage—rescue of irrigated regions now in distress from water shortage by providing them with supplemental supplies—the National Reclamation Association in its meeting last week made a contribution to sound thinking on a matter of vital national concern. It expressed an eminently timely view of the situation, one that agrees fully with the direction of the Bureau of Reclamation's efforts during the past decade.

The irrefutable need for salvage work also constitutes effective answer to the prairie-state argument, in which the Department of Agriculture has traditionally joined, that reclamation should stop until the farm surplus, a prairie-state phenomenon, is done away with. That would mean stopping it forever! The argument is narrowly sectional, in fact, being framed to serve the interests of one part of the country even to the extent of demanding that another part, the arid region, shall stop its further development.

As was brought out at the Salt Lake City meeting, reclamation is essential to the existence of the eight mountain and intermountain states, which make up more than one-third the area of the United States. What it has done for that region, and through it for the country at large, may not be disregarded or undervalued. Though the growth of the reclamation system has not been free of faults, political and other, the need for going on with the reclamation program is fundamental. The constructive line of advance is in the direction of future planning that will take full cognizance of existing conditions in the parched western deserts and utilize past experience to shape sound procedure.

Inadequate water supplies of lands already in cultivation need to be augmented by storage. Where submarginal lands have been developed and their production level cannot be raised, they should be replaced by better areas. Small projects must be undertaken, to utilize supplies of small volume and limited areas of good valley lands. More effective operating methods for reclaimed land need to be developed. These are among the elements that call for present consideration with aid of the best knowledge that can be brought to bear on them.

Earnest, constructive effort to utilize and supplement what we already have is the need of the hour in going forward with reclamation development. The agriculture that the West needs for its subsistence should not alone be preserved but made secure against disaster by full conservation of the remaining water and land resources of the region. In working toward this objective through wise planning for utilization of those resources, and at the same time working to place reclamation on an unassailably sound business basis, western engineers and students of reclamation can give aid to the nation's continued healthy growth. To this end serious technical and economic study must replace propaganda enthusiasm, and the public interest must be the goal of all effort.

CURRENT NEWS

Buffalo Sewer Authority Appoints New Engineers

The Buffalo, N. Y. Sewer Authority on Nov. 13 rescinded its previous appointment of Fuller & McClintock, New York City, as consulting engineers on the city's $15,000,000 PWA sewer and sewage disposal project and by a 3 to 2 vote authorized the appointment of Clyde Potts, New York, as engineer, and retained the firms of Whitman Requardt & Smith, Baltimore, and Greeley & Hansen, Chicago, as engineering consultants. This action followed the breaking of a deadlock in the authority when Mayor George J. Zimmerman appointed Robert Boasberg, his secretary, to an existing vacancy on the five-man board. The new appointee fills a position that has been vacated by three resignations since the authority was organized in July.

The contract was awarded to Mr. Potts on the basis of 3 per cent for design and ¾ per cent for supervision of construction. In addition to his work on the disposal plant and intercepters he may be required to supervise and inspect the construction of stormwater drains to be built by the city's department of public works, receiving for such service cost plus 25 per cent. The two consulting firms will each be paid $100 per day of service with a guarantee of not less than 100 days. As the work is to be done under a federal grant and loan, approval of the engineering contract by PWA is required. At this writing (Nov. 20) such approval has not been given and press reports from Buffalo state that the PWA has been asked to hold up award pending an investigation.

Disagree on cancellation clause

Created by act of the 1935 state legislature, the Buffalo Sewer Authority is charged with full responsibility for the construction of a municipal sewage disposal plant and additional sewerage facilities, to be financed by a federal grant of $6,750,000 and a loan of $8,250,000. The selection of an engineer for this work was made on Sept. 12, when Fuller & McClintock were appointed from a group of 24 engineering firms who had been asked for proposals and statement of qualifications. Execution of a contract with them was delayed, however, by difficulty in agreeing on the form of a cancellation clause. Settlement of the difference of view was momentarily expected when the board took the action above reported.

Within the Authority there have been differences regarding the plan of procedure for the design and construction of the disposal plant and the new sewerage facilities. Two members advocated a unit contract form which provided that the consulting engineer would prepare complete plans and assume full responsibility for the successful completion of the disposal works and the sewers. The others favored a contract under which the Sewer Authority and the consulting engineer were to deal only with the disposal plant while the sewer was to be designed and constructed by the city's department of public works.

National Reclamation Association Considers Western Irrigation Problems

THE fourth annual conference of the National Reclamation Association, held in Salt Lake City Nov. 15 and 16, reflected decided increase of interest in its work by larger and more representative attendance than previous meetings have enjoyed. About 250 were present at the opening session and the considerable number of members of Congress, high officials of state and federal departments and representative leaders in reclamation gave weight and significance to the proceedings. The program was heavy with papers and the two days, including the intervening evening session, were fully occupied with their presentation.

At the opening session, after a cordial welcome by Henry H. Blood, Governor of Utah, Marshall N. Dana, of Portland, Ore., president of the association, outlined some of the fundamentals of the problems to be considered. He pointed to the probability of a population increase in the United States of 15,000,000 to 17,000,000 in the next 25 years and the consequent importance of increased areas of irrigated land and more effective agricultural production. Land reclamation must be considered in relation to other industrial factors and action should be taken by the association in establishing policies.

Although Elwood Mead, Commissioner of Reclamation, could not be present he sent as his personal representative B. E. Stoutemyer, Portland, Ore., district counself, Bureau of Reclamation, and arranged for other officials of the bureau to attend. A paper on "The Emergency of Reclamation" was presented by Mr. Stoutemyer. In a letter to the meeting expressing encouragement, Dr. Mead spoke enthusiastically of the future of reclamation.

Seven-point program proposed

L. C. Gray, Chief, U. S. Resettlement Administration, in a paper on The Nation's Land Problem, suggested a seven-point program for reclamation policy including among the elements "complete divorcement of reclamation from politics through control by a non-partisan board," putting drainage and irrigation under the same direction, a low interest rate to avoid interest exemption evils, and due allowance for costs properly allocated to power, flood control and other elements in multiple-purpose projects.

Opposition views appeared promptly not only in the comment of John W. Haw, director of agricultural development, Northern Pacific Ry., who led the discussion, but also in the comment of other speakers. Mr. Haw said the West does not oppose federal appropriations for improvement in eastern states, disagreed with Dr. Gray as to conditions in the West, pointed out that there has been undue emphasis on pleas for moratoriums, and urged that western farmers be not forced from irrigation projects into industrial districts of the Pacific Coast or to the middle west. George W. Malone, of Nevada, cited conditions in his state which make local production essential regardless of overproduction in areas from which transportation costs would be prohibitive.

Prof. E. Grosvenor Plowman, University of Denver, in a paper on Economics of Water Development in the Upper Basin States outlined problems arising from the fact that in the eight Rocky Mountain states, which comprise one-third the area of the United States, the total population is only five million people. This relatively uninhabited region is almost wholly dependent upon reclamation. In the four upper basin states, with only 2,000,000 people and with 6,000,000 irrigable acres, half the area is government land—an argument for generous appropriation of federal funds. He stressed the need for a regional planning board to coordinate plans for future development of this area.

Land and Water Planning was the subject of a paper by Judge Robert W. Sawyer which outlined the difficulties of long-range planning in an age when extensive changes occur frequently. Judge Sawyer summarized his observations by the conclusion that it would be best to adopt the recommendations of the Haw-Schmitt report on Federal reclamation policy.

National reclamation policy

A summary of the Haw-Schmitt report was presented by George O. Sanford, general supervisor of operation and maintenance, U. S. Bureau of Reclamation. At the evening meeting, presided over by Governor Blood, President Dana read a paper by F. E. Schmitt, editor, *Engineering News-Record*, which discussed reclamation economics and made suggestions on future matters of policy supplementary to the Haw-Schmitt report.

A comprehensive explanation of what it is proposed to accomplish by the California Central Valley project was given by Edward Hyatt, state engineer of California. F. E. Weymouth, chief engineer and general manager, Metropolitan Water District of Southern California, described the Colorado River Aqueduct in its relation to Boulder Dam, and F. A. Banks, U. S. Bureau of Reclamation project engineer at Grand Coulee dam, gave an outline of the economics and essential features of that undertaking. Prof. Harlan H. Barrows, University of Chicago, representing the National Resources Committee, outlined the work that has been done by the water resources division, with special reference to reclamation.

Officers elected for the ensuing year were: president, O. S. Warden, Great Falls, Montana; first vice-president, Roland Harwell, El Paso, Texas; second vice-president, H. Lloyd Miller, Sunnyside, Wash.; secretary-treasurer, L. A. Campbell, Helena, Montana.

Contractor Abandons German Sheet Piling for New York Bridge

FOLLOWING a storm of protest over the intended use of Kloeckner steel sheet piling, made in Germany, for sheeting the sides of a depressed roadway section of the Queens approach to the Triborough bridge in New York City, the Woodcrest Construction Co., contractor for the work, has decided to substitute American-rolled sheeting despite the contention that the German pile section is better suited to this particular installation. The Kloeckner pile is deep arch Z-section with interlocks at the center of the arch span instead of at the corners, the conventional American design. It is rolled in sections with a section modulus as high as 62, whereas the heaviest domestic pile has a section modulus of 15.3. It is reported that a Kloeckner pile with a section modulus of 28 was to be used, providing a much higher beam strength than that of the heaviest American pile.

The depressed roadway, 110 ft. wide, extends from 29th St. to 70th St. in Astoria. At the deepest point the subgrade is 27 ft. below street level. Water level is about 12 ft. below the street, and below the water table lies a deep layer of very wet, fine sandy silt, known locally as "bulls liver." The roadway is to be covered by a 3½-ft. concrete slab, the bottom of which is about 17 ft. below the water table. Integral with the slab are heavy concrete retaining walls extending to the top of cut.

The plans call for permanent sheet piling walls to be driven from water level to 10 ft. below bottom of slab to cut off the water pressure from the sides beneath the slab and to minimize the possibility of mud boils and loss of ground in the slab foundation. The contractor considered the Kloeckner piling ideal for this work, counting on the high beam strength to provide a cantilever retaining wall above the slab during construction operations. The central section of the slab was to be built first, then raker braces were to be installed from the top of the piling to the center concrete, supporting the sheeting while the outside sections of the slab is built.

Some 500 tons of sheeting are involved. Upon learning that the contractor had placed an order for the Kloeckner piling, protests to PWA, which is financing the bridge, and to the Triborough Bridge Authority poured in from American steel manufacturers, the American Federation of Labor, the Mayor of New York and from many other sources. At the time of the purchase, PWA rules permitted the use of foreign materials providing the price was 15 per cent below that of domestic products. Effective Nov. 19 the differential was raised to 25 per cent.

Meanwhile, President Roosevelt has requested the Federal Trade Commission to investigate whether identical bids by four domestic firms for piling on the Morehead City project resulted from collusion.

The contractor reports that the delay in placing the order for the foreign sheeting has made it impossible for the mills abroad to furnish the steel as soon as needed. Desire to proceed with the excavation work at once is given as the main reason for resorting to domestic sheeting.

Herring Becomes Director Of A.M.E. and I.A.P.W.O.

Frank W. Herring, formerly assistant director of the Joint Secretariat of the American Society of Municipal Engineers and the International Association of Public Works Officials, has been appointed as executive director by the joint administrative board to succeed Donald C. Stone whose resignation became effective on Oct. 16.

Prior to his work with the Joint Secretariat staff, Mr. Herring was for five years on the editorial staff of the *Engineering News-Record* and before that was with the engineering division of the Detroit Department of Water Supply.

Mr. Stone resumes his full time duties as executive director of the consulting and research division of Public Administration Service in Chicago. From March until October Mr. Stone has been engaged in advising the Works Progress Administration in its work of establishing administrative procedures in connection with the federal works program.

R. W. Lesley, Pioneer in Cement Industry, Dies

Robert Whitman Lesley, director of the Giant Portland Cement Co., and a pioneer in the development of the portland cement industry in this country, died at his home in Haverford, Pa., Nov. 10, at the age of 82.

Mr. Lesley attended the University of Pennsylvania for a time and then joined the staff of the Philadelphia *Public Ledger* and later became assistant editor. At the same time he was studying law and was admitted to the bar in 1879. While still working in the newspaper office he accidentally became interested in the manufacture of portland cement when, during the panic of 1873, he found himself pledged to sell two carloads of cement in order to assist the cement company which was owned by his uncle. With J. W. Trinkle, Mr. Lesley formed the firm of Lesley & Trinkle, and after continuing for some years in both the editorial and cement business, the partners were able to buy out the firm of Campbell, Harris & Co., thus becoming one of the leading cement companies in Philadelphia. During this time a number of experiments were undertaken which resulted in the patents covering many of the processes now in use in the manufacture of portland cement in this country.

In 1883 Mr. Lesley founded the American Cement Co. and after hard work managed to build up a reputation for American portland cement against the better known foreign brands.

Mr. Lesley kept his interest in the scientific side of portland cement production and in 1906 gave funds for the equipment of a complete cement laboratory to the University of Pennsylvania. Mr. Lesley at one time was vice-president of the American Society of Testing Materials as well as an associate of the American Society of Civil Engineers, a member of the American Chemical Society and a member of the National Advisory Board on Fuels and Structural Material as well as being an active member in many other societies.

President Roosevelt Revises Procedure For Central Valley

(Washington Correspondence)

PRESIDENT ROOSEVELT has complied with the request of Sen. Johnson, Calif., for another revision of the work schedule on the $167,000,000 Central Valley conservation project. The President's order modifying distribution of the $15,000,000 allotted for the first year's work has not been made public but, according to Sen. Johnson, now includes $5,000,000 for commencing on relocation of the line of the Southern Pacific R.R. The Reclamation Bureau is ready to proceed as soon as officially informed of the change in plans.

The new plan of procedure ignores the policy that expenditure of work relief funds must produce a useful unit. This may cause trouble with the General Accounting Office but to clear funds for the project reliance is placed on the fact that contracts awarded this year, even though they do not complete any work unit that is independently useful, will not actually obligate more funds than have been allotted.

The letter of President Roosevelt to Senator Johnson reads:

"I have received your letter of Oct. 9, in which you discuss conditions affecting the development of the Central Valley project in your state and express the belief that a modification of the statement approving the allotment of $20,000,000 should be made. The need for this modification arises, in part, out of the fact that the allocation was reduced from $20,000,000 to $15,000,000, which reduction prevents carrying out the original plan.

"As I understand it, the modification is needed for the important reason stated in your letter—that it is a more logical arrangement for beginning the development of this important and complex project and will better satisfy the experts and the people of the state vitally concerned in this conservation program. After careful consideration of the statement in your letter I have decided to concur in your recommendations and have modified the language of the allotment of $15,000,000 to conform to your recommendations. A copy of the modified allotment is enclosed.

"I realize that the basis of this development is the conservation of the waters of the Sacramento and San Joaquin rivers and that through such conservation there will be a great increase in the volume of water available for irrigation, domestic uses and the generation of hydro-electric power. I am satisfied that the demand both for water and cheap power in California is so great that no difficulty need be anticipated in finding a market for both as soon as development has reached a stage where their delivery is assured and contracts with definite terms are possible. Hence, there is no need for the immediate making of contracts entailing great delay in beginning construction and employment of labor.

"I therefore approve the beginning of construction before these contracts are executed. Knowing the value of this project to California and the anxiety with which you await the initiation of this work, it is a great satisfaction to comply with your request."

New Jersey Sanitary Association Discusses Mosquito Control and Water Conservation

MOSQUITOS in the role of friends as well as enemies of man was one of the interesting revelations made at the mosquito control session of the New Jersey Health and Sanitary Association which met in Asbury Park on Nov. 15 and 16 for its 61st annual meeting. The engineering session of the association was devoted to water supply conservation and a discussion of a new odor-reducing compound tried experimentally at a local sewage plant.

In pointing out mosquito hazards in reservoirs and other impounded waters, James E. Brooks, consulting engineer, prefaced his remarks by stating that the very presence of these insects in a particular locality indicated that all was not well. Mosquito larvæ are natural scavengers which feed upon and remove organic pollution in water and thus they perform a useful function. However, in the adult stage they are not only a nuisance but in some cases propagate disease. If reservoirs and lakes are to be free from breeding mosquitos, they must be free of pollution, and it is essential that the vegetation, both floating and rooted varieties, be removed so that larvæ cannot find refuge from fish. Polluted water not only kills fish life but attracts and supports the breeding of mosquito larvæ in enormous numbers.

Better and more economic mosquito control resulted from the installation of adequate sewerage facilities in Union County, according to R. J. Van Derwerker, superintendent of the mosquito commission in this district. In one community alone, $3,000 a year was appropriated annually for oiling in a vain effort to control mosquito breeding in a neighboring stream that received the effluent from a poorly-operated sewage disposal plant. When this plant was abandoned two years ago and the town connected its sewerage system into a county-wide trunk sewer, it was possible to discontinue the extensive oiling operations and mosquito activity was automatically curtailed. It was stated that while many varieties of mosquitos would breed prolifically in sewage-diluted waters, only a few species could breed in raw sewage itself. Carbon dioxide gas, an accompaniment of organic putrefaction, incites and promotes physiological activity in mosquitos and has been shown to be a potent factor in their selection of sewage-polluted water for propagation.

The possibility of malarial outbreaks, according to Dr. T. J. Headlee, state entomologist, hinges upon the presence of the *Anopheles Quadrimaculatus* species of mosquitos who may chance to bite persons with the malarial parasite in their blood. Since it is impossible to control the movement of human carriers of the disease, the only practical way of coping with the problem of reducing malarial outbreaks is to destroy the mosquito vector by eliminating breeding places. In New Jersey, where active control measures have been under way for many years, malaria is a steadily diminishing disease. The annual number of deaths per year has been reduced from 330 in 1879 to the yearly average of 3.1 during the last decade.

The seriousness of a diminishing water supply in New Jersey, where no comprehensive plan exists for the conservation or development of water resources, was outlined by Max Grossman, chairman of the State Water Policy Commission. According to Mr. Grossman, his group is charged with the duty of conserving and allocating water but the only appropriation ever made to develop plans was diverted to other uses. Limited funds and little authority has been delegated to the commission and it is practically helpless in formulating definite policies. He made a plea for unified action of all those bodies within the state that are concerned with purveying water, so that appropriate legislation can be passed to create an authority that will be empowered to study and administer the problems of water conservation. An acute situation faces the northern metropolitan section of New Jersey where forecasts have indicated that new supplies are definitely needed by 1943. Business revival in this highly industrialized area, it was warned, will create demands that will bring a recurrence of the situation in 1929 when many communities were forced to ration their water and in so doing severely curtailed promising industrial developments.

Auto transportation is facilitating a wide public activity in outdoor recreation and this has introduced new hazards to formerly inaccessible watersheds in the form of increased pollution entering tributary streams that are adjacent to reservoirs. M. W. Cowles, health officer of the Hackensack Water Company, in outlining the broad aspects of watershed protection, stated that in order to cope with this new problem of recreational usage, the two factors of primary consideration were adequate provision for the disposal of human wastes and the control of bathing. The laws of New Jersey are not clearly defined in regard to standards for bathing in portable waters, but the establishment of cordial relations between water purveyors, property owners, and the health authorities has resulted in voluntary agreements to prevent pollution.

Studies made at the Freehold, N. J., sewage treatment plant in odor control, using chlorinated hydrocarbons (cloroben) were described by W. J. Schiverea, borough engineer. Strong septic sewage, averaging 600 p.p.m. suspended solids, enters the plant from three pumping stations and pre-chlorination has proved ineffective in dissipating the disagreeable odors at the plant inlet. Cloroben applied in the pump wells reduced hydrogen sulphide formation in the sewer line and modified odors. It did not have any effect on the bacterial action of the trickling filter or the sludge digestion process. Present indications are that this is a satisfactory deodorant around the plant but complete treatment of the entire sewage flow would be expensive.

Plans of the Interstate Sanitation Commission were discussed by William C. Cope, member of the New Jersey group, who related that a $350,000 request to PWA for carrying out sanitary surveys had been rejected. Application has now been made for $50,000 to begin studies that will eventually result in sanitation improvements costing New Jersey $25,000,000 and New York $300,000,000.

Reclamation Bureau Calls Bid For Construction of Projects

With most of its projects under construction or advertised for bids, the Bureau of Reclamation has advertised for bids on additional work, leaving only six projects not yet under way. Bids will be opened on Dec. 2 for the following projects in the bureau's $81,150,000 program:

Earthwork and tunnels on the main canal, Gila Valley project, Yuma, Ariz.; Tunnels Nos. 1, 2, 3 and 4, Black Canyon Canal, Payette Division, Boise project, Ontario, Ore.; Tunnels 1, 2 and 3, Yakima Ridge Canal, Roza Division, Yakima project, Yakima, Wash.; Tunnels 1, 2 and 3, Shoshone Canyon Conduit, Heart Mountain Division, Shoshone project, Cody, Wyo.

Bull Lake Dam on the Riverton project, Wyo., will be advertised for bids about the same time bids on the other projects are received.

Work is now underway on Bitter Root, Frenchtown, Owyhee, Vale, Moon Lake, Ogden River, Grand Coulee and Casper-Alcova. Instead of being advertised for contract, work will be done by force account on the Klamath drainage project, Belle Fourche, Arrowrock Dam repairs, the Boise project and the Colorado River of Texas. It is expected that specifications will be ready in the very near future on Central Valley, Calif., Alma Gordo Dam, N. M., Provo River, Utah; and the Yakima storage project in Washington, and plans for the Deschutes River project, Ore., are in suspense. Bids on Salt River, All-American Canal, Sun River and Unity Dam have already been asked and those for Unity Dam on Burnt River, Ore., will be opened at Vale, Nov. 30. This is an earth fill structure for which an allotment of $500,000 has been made.

Charles E. Crownover, of Yakima, Wash., has been appointed as construction engineer of the Roza Division of the Yakima reclamation project, for which an allotment of $4,000,000 was made. Mr. Crownover, who will have charge of the construction for the Bureau of Reclamation, formerly was a member of the bureau's engineering staff. He served the bureau for about 15 years, leaving it to become city engineer of Yakima. While employed by the bureau, he was in charge of construction of Keechelus Dam, one of the storage dams serving the Yakima project.

Prevailing Wage Rates to be Paid to Passamaquoddy Workmen

As a result of an order issued by WPA Administrator Hopkins, the Passamaquoddy tidal power project has been exempted from the general relief wage scale until Jan. 1 in order that homes for the 1,200 administrative staff members might be completed before cold weather sets in. In order to help speed the work, a 45-day state of construction emergency was declared allowing Lt. Col. Philip B. Fleming of the Corps of Engineers, to raise the prevailing hourly wage rate above those ordinarily paid to relief workers in order to obtain more efficient workmen. The new scale will pay common labor 40c. an hour and skilled labor 75c. an hour. The new scale, in effect, will more than double the wages of all classes of workers.

State Engineers Of West Discuss Underground Water

PROBLEMS relating to underground waters in their legal, regulatory and conservation aspects occupied the attention of state engineers and members of their staffs who met in Salt Lake City, Nov. 13 and 14, for the eighth annual meeting of the Association of Western State Engineers. The states represented were Utah, Nevada, Arizona, New Mexico, California, Idaho, Wyoming and Montana. In the address of welcome, Henry H. Blood, Governor of Utah, pictured the challenge to engineers that appears in a situation where so much pioneering yet remains to be done to develop water supplies, and to provide flood and drought protection. He urged them to take more active leadership in a world sadly in need of rehabilitation.

Artesian well control

Thomas M. McClure, state engineer of New Mexico, presented an outstanding paper on underground water problems, particularly the control of artesian wells. Discussion of this topic occupied almost the entire first day and consisted of an exchange of experience in western states generally. Regulatory provisions differ in the several states but there was agreement on the importance of studying underground waters and providing for more adequate and effective control. The common opinion was that the sooner all states have effective laws concerning underground waters the less trouble there will be on this score.

Utah has provided for regulation of underground waters without special legal enactment by making all waters of the state subject to the same laws of appropriation and use. New Mexico has been notably successful in preventing waste of underground waters by plugging artesian wells not in use. In the four years since this conservation work has been under way the cost of such work has been reduced 75 per cent by improvement in methods and equipment.

Snow surveys

Snow surveys as developed in Nevada and now used extensively in many states were discussed with emphasis on improvement in equipment and technique. Snow samples now are taken on predetermined lines protected from drift and wind and above the winter melting line. Utah's snow surveys for a runoff on the order of 130,000 acre-ft. can be depended upon, it was said, to predict total runoff within 10 per cent of total volume. Last year these predictions gave 6 to 8 weeks drought warning and thereby materially aided in avoiding a $5,000,000 crop loss.

Authority was voted to the executive committee of the association to act in developing closer contact with the National Reclamation Association with the purpose of making the work of both organizations more effective. Officers elected for the ensuing year were president, Thomas M. McClure, state engineer of New Mexico; vice president, James S. James, state engineer of Montana; the third member of the executive committee, T. H. Humpherys, state engineer of Utah.

Control Gates in Chicago River Requested by Sanitary District

In a hearing Nov. 15 before Capt. S. N. Karrick, acting district engineer, U. S. Army, Chicago, the Sanitary District of Chicago, presented a request for a permit from the War Department to build control works in the Chicago River between Michigan Ave. bridge and the Outer Drive bridge. The purpose of the two flood gates would be to prevent the reversal of the Chicago River in times of heavy rainfall and the consequent discharge of sewage polluted river water into Lake Michigan with the possibility of contaminating the municipal water supply.

Alternate plans for an outer harbor lock and barge terminal were presented by Hugh E. Young, chief engineer, Chicago Plan Commission. Shipping interests and dock owners protested against the district plan which was estimated to cost $800,000 to $1,000,000 as against $4,800,000 for the outer harbor plan. The district indicated a willingness to contribute to the latter but William A. Rowan, alderman, said the city could not provide its share of the additional funds. Robert I. Randolph, president, Mississippi Valley Association, requested delay with the hope that Congress could be induced to defray the additional costs.

After 1938 when the diversion of lake water is cut to 1,500 sec.-ft. the velocity in the river due to the net average of 950 sec.-ft. possible to divert directly will be 0.16 ft. per second or 10 ft. per mile. The reversal of such a slow current could easily happen during a time of heavy rainfalls. At present rainfalls of sufficient intensity to reverse the flow occur ten to twelve times each year.

The district has obtained PWA funds to install gates but the agreement with PWA has a time limit. The PWA engineering review board studied several plans and directed that the district submit these particular gate-control plans.

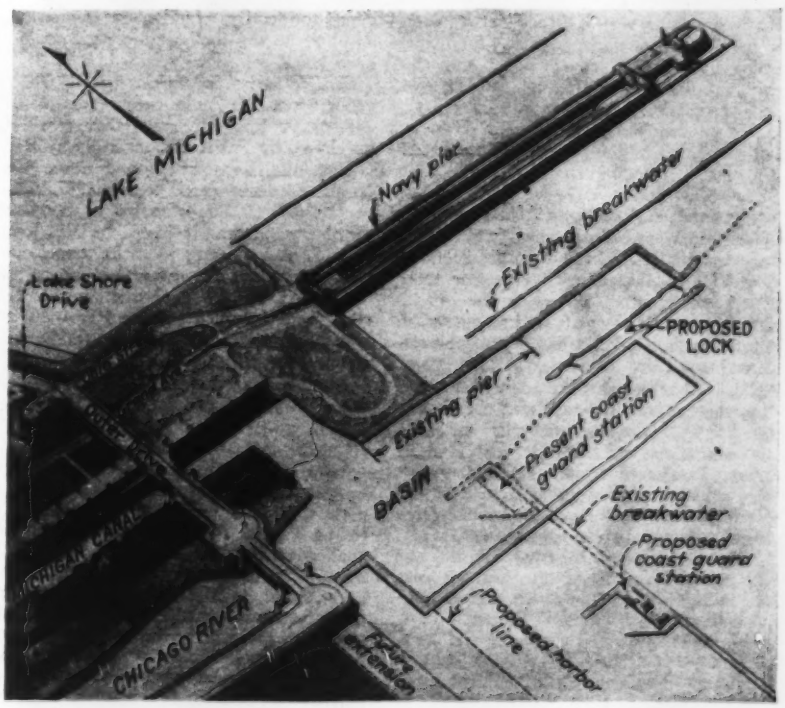
The city has long had in mind the installation of fixed bridges over the Chicago River, the first step toward which is an outer harbor for barge interchange so that masted ships would not need to come into the river. At present no means of financing the harbor even with Sanitary District contribution is in sight.

Captain Karrick indicated that he would recommend controlling gates and asked that detailed plans be submitted at an adjourned hearing Dec. 13, such plans to be made available to interested objectors 10 days prior to the hearing, at his office and that of the district.

Tennessee Contractors Plan for Building Industries Council

Tentative plans for a Tennessee Building Industries Council, to protect the members of the construction field from another "1929-1934 catastrophe" were completed at a joint meeting, Nov. 15, Memphis, Tenn., of 200 architects, contractors, subcontractors, and building material dealers of Tennessee. The proposed plan for the new organization is to have a delegate council, with a delegate from each division of the building industries. This council would meet once a year at the same time and place where the conventions of the other organizations in the industry convene.

Stephen F. Voorhees, president of the American Institute of Architects, urged the convention to formulate a plan for such an organization and told them to prepare for ten years of good business, stating that business revival is on the way.



AN OUTER HARBOR at the mouth of the Chicago River is proposed by the Chicago Plan Commission in preference to the installation of control gates in the river.

New Jersey Coast Damaged By Severe Northeast Gale

General damage to beaches and beach front structures by the northeast gale of last Sunday (Nov. 17) is reported in telegrams from New Jersey coast towns from Atlantic Highlands to Cape May. For the most part the damage was in detail to old bulkhead walls, jetties and boardwalks although some recreation piers were damaged at Asbury Park and other resorts, and sewage works at Deal were damaged so as to require extensive repairs.

The most general damage was shore erosion which practically eliminated the beaches in places, undermined beach front roads, exposed sewer and water lines and covered the foreshore with tons of sand, bulkhead rocks and jetsam of all kinds.

High tides piled up by the wind flooded low-lying land, streets and buildings.

Estimates of the damage range from one to several million dollars but all are quite uncertain until a complete survey is made and this is now being undertaken by the army engineers. Structures built according to standards approved by the New Jersey Board of Commerce and Navigation suffered little damage.

Shore muncipalities generally have appealed for federal aid to repair damages and particularly to undertake a general program of works to prevent continued shore erosion and beach denudation.

Port Authority Rejects Bids For Midtown Tunnel Work

Proposals, opened on Nov. 7, for the construction of the Weehawken land sections and plaza of the new Midtown Hudson Tunnel have been rejected by The Port of New York Authority and new bids will be solicited later this month. The decision to reject and to readvertise was reached by the commissioners following receipt of detailed advice from the engineering department that all figures previously submitted were greatly in excess of the engineers' estimates. No delay in the completion of the project is anticipated as a result of the bid rejection.

The construction included in the New Jersey land sections and plaza will be divided into two contracts, and other changes will be made in the specifications in order to obtain a more favorable total price. One contract will cover the 785 ft. of rock tunnel excavation running west under King's Bluff from the present ventilation shaft to a point near the plaza portal. The second contract will include all open excavation work and the laying out of the plaza. The latter also embraces the relocation of 3,000 ft. of Hudson County Boulevard East. The plaza contract will not include the superstructure of any of the buildings that will be required for operation of the tunnel.

The bids rejected were as follows, the first figure representing the cost of cast iron lining for the tunnel and the alternative being for structural steel:

George M. Brewster & Son, Inc., Bogota, N. J., $3,875,528.50 and $4,360,588.50; Underpinning Foundation Co., New York, $4,511,453 and $4,493,373; joint bid from the Necaro Co., Inc., and the Arthur A. Johnson Corp., Brooklyn, $4,600,212.00 and $4,579,702.00.

A.G.C. Convention at Miami; Klinger to be New President

The Associated General Contractors of America have selected Miami, Fla., as the location of the 1936 annual convention, to be held at the Miami-Biltmore hotel Jan. 13-15. Wm. A. Klinger, president of W. A. Klinger, Inc., building contractor of Sioux City, Ia., has been nominated as president of the organization for next year, succeeding Nick Helmers, of St. Paul, who has held the position for the past two years. Edward P. Palmer of Senior & Palmer, Inc., New York City, has received the nomination for the new vice-president at large, to succeed A. W. Merchant, of Providence, R. I. Tentative plans for the convention include discussion of anticipated national legislation affecting contracting and questions concerning continued federal relief works.

RFC Offers To Finance Part Of East River Tunnel

A proposal for financing construction of the projected 38th Street tunnel under the East River by a government loan of $58,000,000 to New York City has been discussed with President Roosevelt by Sen. Wagner, New York. As a self-liquidating project, the tunnel could be financed through the Reconstruction Finance Corporation, in the senator's opinion. He has called on Jesse Jones, chairman of RFC, also upon Harold Ickes, PWA administrator with reference to the project, which he believes will be necessary to handle crowds attending the world's fair on Long Island in 1939.

Construction might be financed jointly by PWA and RFC as Mr. Ickes said that the amendment of a contract for another New York project, which he declined to identify, would release some funds and that he was looking into what could be done about it.

Sen. Wagner is seeking a $11,000,000 grant from PWA, in conjunction with a $47,000,000 loan on bonds that would be purchased by PWA and taken up by RFC.

That the Reconstruction Finance Corporation was prepared to loan $47,130,000 of the $58,380,000 estimated as necessary for the tunnel was indicated by Mr. Jones, Nov. 7, at a meeting with the tunnel commissioners. At the same time he pointed out that it was doubtful whether the RFC could loan the money if the PWA did not make a grant of $11,250,000 to aid in the construction of the project.

After having discussed the project with the Queens Midtown Tunnel Authority, consisting of Alfred B. Jones, Albert T. Johnston, and William H. Friedman, Mr. Jones said that the funds could be provided by the Reconstruction Finance Corporation by its buying tunnel bonds sold to the PWA. The authority has already applied to the PWA for a loan of $43,224,000 and a grant of $15,141,000.

According to Mr. Jones, RFC plans and traffic estimates would have to be checked to determine whether or not the tunnel would be a self-liquidating project. He indicated than an answer could be given in two weeks. The tunnel authority reported that preliminary plans are well under way and that a final report would be ready in three months.

Construction League Policy Committee Urges Ending WPA

A DEFINITE STAND against WPA construction as means of relief and a favorable attitude towards continuation of federal aid for highways and PWA assistance to state and municipal public works was taken by the Construction League of America at the meeting of its Policy Committee on Nov. 8. The Committee also urged that projects financed by federal funds be carried out in normal manner by contract. The statement of the Policy Committee follows:

The Construction League recognizes the need for the utmost reasonable economy in public administration in order to prevent excessive tax burdens and maintain the public credit and yet avoid any drastic increase in unemployment. The greatest opportunity for economies naturally will be found in connection with administrative and other overhead costs of government, but it is evident that the investment of the public credit in public works and facilities should be conducted with due regard to economy, efficiency and the permanent values to be realized by the community.

Public works program

Accordingly, the Construction League recommends the following principles to govern the conduct of federal, state and local public improvements:

1. Elimination of the wasteful construction projects now carried on by WPA as a means of relief.

2. Continuation of a substantial program of useful federal public works and of federal aid to highways.

3. Continuation of PWA aid to states and municipalities for useful projects that add to the public wealth.

4. Grants for state and municipal projects and emergency appropriations for highways to be on a descending scale in proportion to the increase in private employment.

5. Ear-marking of all appropriations for particular purposes.

6. Mandatory provisions that construction projects financed in whole or part by federal funds should be carried out in a normal manner through public lettings and contracts and the regular engagement of professional services without arbitrary and artificial restrictions.

Denies promise of cooperation with Berry

The Construction League has denied newspaper reports that the League had promised 100 per cent cooperation in the industrial clinic called for Dec. 9 by George L. Berry, the Coordinator for Industrial Cooperation.

It was pointed out that no official action had been taken by the League assembly and that such assurances of cooperation had been given to Coordinator Berry by a number of leaders of the branches of the construction industry in their individual capacities.

The Construction League, however, is urging members of its various constituent associations to attend the December conference.

PWA Housing Faces Delay On Boston Project

PWA has been ordered to show cause in the Supreme Court of the District of Columbia on Nov. 25 why a temporary order should not be issued restraining the Housing Division from proceeding with its $6,000,000 Old Harbor Village project in Boston. The action was filed by six property owners in the neighborhood who allege that the project will destroy the value of their investments. The plaintiffs challenge the constitutionality of law under which PWA's low rent housing program is carried on, alleging that they are being deprived of their property without due process of law.

Commenting on the suit for injunction, Administrator Ickes said the facts will show that in all cases where PWA locates a housing project the effect will be to increase the value of adjacent property.

"Every project is providing a powerful stimulant to real estate activity in areas which were moribund," according to A. R. Clas, Director of the Housing Division. He cited Atlanta, where two projects now are nearing completion. In the Techwood area, he said, five large lots were sold in a rising market. Within a month after construction began on the federal project, in the University area, a real estate corporation purchased a whole block for residential development.

SOCIETY CALENDAR

HIGHWAY RESEARCH BOARD, annual meeting, Washington, D. C., December 5-6.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, annual convention, Miami, Fla., December 9-12.

AMERICAN SOCIETY OF PLANNING OFFICIALS, state planning conference, Chicago, Ill., December 12-13.

NATIONAL RIVERS & HARBORS CONGRESS, special meeting, New Orleans, La., December 13-14.

NATIONAL ASSOCIATION OF HOUSING OFFICIALS, annual meeting, New York, December 14.

AMERICAN ENGINEERING COUNCIL, annual meeting, Washington, D. C., January 9-11, 1936.

ASSOCIATED GENERAL CONTRACTORS OF AMERICA, annual meeting, Miami, Fla., January 13-15, 1936.

AMERICAN ROAD BUILDERS' ASSOCIATION, annual convention and road show, Cleveland, Ohio, January 20-24, 1936.

CONSTRUCTION EQUIPMENT ASSOCIATION, first annual meeting, Cleveland, Ohio, January 20-24, 1936.

ASSOCIATION OF HIGHWAY OFFICIALS OF NORTH ATLANTIC STATES, annual meeting, Atlantic City, N. J., February 12-14, 1936.

AMERICAN SOCIETY FOR TESTING MATERIALS, regional meeting, Pittsburgh, Pa., March 4, 1936.

NEW YORK STATE HIGHWAY CHAPTER, Associated General Contractors of America, tenth annual convention, Albany, N. Y., December 4-5.

WASHINGTON IRRIGATION INSTITUTE, annual meeting, Wenatchee, Wash., December 12-13.

NEW YORK SECTION, AMERICAN WATER WORKS ASSOCIATION, midwinter meeting, New York City, December 27.

EXAMINATIONS for registration of professional engineers will be held in the near future as follows:

IOWA—Examinations for registration by the Iowa State Board of Engineering Examiners will be held at the State House, Des Moines, Iowa, December 3-5, 1935.

Personals

S. LOGAN KERR has recently been appointed senior engineer in charge of the mechanical engineering section for the Passamaquoddy project now being constructed by the Corps of Engineers at Eastport, Me.

J. H. CUNNINGHAM, of the Huntington, W. Va., district U. S. engineers staff, has been transferred from the Gallipolis (O.) dam site to the Winfield Dam, on Kanawha River.

JOE R. JONES, formerly county engineer at Kingfisher, Okla., has now taken a position in the engineering department of the Soil Conservation Service, U. S. Department of Agriculture, with headquarters at Colorado Springs, Colo.

JOHN A. GILMAN has been transferred to the U. S. Military Academy at West Point, N. Y., to supervise a $5,474,250 construction program. Capt. Gilman recently supervised the construction of improvements at Fort Knox, Ky.

B. J. LAMBERT, head of the civil engineering department of the University of Iowa, has been named acting dean of the university's engineering department, succeeding C. C. Williams, who recently accepted the presidency of Lehigh University.

J. K. MCNEILLIE has been appointed resident vice president of the Delaware & Hudson Ry., with headquarters at Montreal, succeeding the late F. P. Gutelius. Mr. McNellie has been with the Delaware & Hudson since 1917 as general superintendent.

JABEZ G. GHOLSTON, chief engineer of PWA inspection division, has been appointed director of engineering for the PWA to succeed Clarence McDonough, who recently resigned to accept a position as general manager of the Lower Colorado River Authority. Mr. Gholston served for two years as inspector and later became senior engineer in the inspection division, and was later made chief engineer. Previous to his government service Mr. Gholston had been active in civil and mining engineering in Mexico and South America, as well as in the United States. He served for a time with the civilian corps of engineers on Mississippi levee work in Arkansas and Mississippi and also with the Baltimore & Ohio R.R. as construction engineer.

J. G. RILL has been made chief of freight transportation of the western division of the Pennsylvania R.R. with headquarters at Chicago. Mr. Rill began first with the Pennsylvania in the operating department of the Maryland division in 1910. In April, 1927, he was made superintendent of the Logansport division and later served as superintendent of the Columbus and the Philadelphia Terminal divisions, and general superintendent of the Eastern Ohio division. Mr. Rill was advanced to his position as general manager of the Chicago division in 1931, and held that position until his appointment as chief of freight transportation. W. D. WIGGINS, chief engineer at Pittsburgh, has been appointed acting chief engineer of the Pennsylvania R.R. Mr. Wiggins is to be succeeded by WILLIAM B. WOOD, who became acting chief engineer at Pittsburgh.

Obituary

JAMES W. HENSLEY, contractor and consulting engineer, Indianapolis, Ind., died at his home on Nov. 6. He was 76 years old.

HARRY G. MENKE, assistant engineer, Alabama Highway Department, died in Carthage, Mo., after a short illness, Oct. 28. He was 44 years of age.

DANIEL J. KILEY, president and general manager of the D. J. Kiley Construction Co., Boston, Mass., died at his home in that city on Nov. 12. He was 61 years of age.

JAMES G. ROBERTSON, president of the Bay State Builders Supply Co., Cambridge, Mass., for nearly 15 years, and formerly connected with Waldo Brothers, Roxbury, Mass., died on Oct. 21 in Washington, N. H. He was 50 years old.

BENJAMIN E. ROBBINS, president of the Robbins Phalon Co., Boston, Mass., representatives of the Dayton Pump & Machinery Co., Dayton, O., and a well known figure in the eastern hydraulic engineering field, died at his home in Milton, Mass., on Nov. 7, at the age of 64.

JOHN T. WALDHAUSER, 65, vice president and superintendent of dredging for the Arundel Corp., Baltimore, died Nov. 11 at his home there. Mr. Waldhauser had been connected with the firm for a number of years and was made vice president this year.

WILLIAM CARLTON READ, for 25 years associated in an official capacity with metallurgical research and development activities of units of Union Carbide and Carbon Corp., died Nov. 6 at his home in New Rochelle, N. Y. He was 47 years old. Mr. Read graduated from Massachusetts Institute of Technology in 1909. In 1910, he became associated with Union Carbide Co. at Niagara Falls, N. Y. Later he was transferred to the Saulte Ste. Marie, Mich., plant and in 1917, returned to the Niagara Falls plants, where he remained until 1928 when he was transferred to the general offices in New York. Since 1930 he had been associated with Dr. F. M. Becket in research and development work.

JOSEPH E. KUHN, Major General in the Corps of Engineers, retired, died in San Diego, Calif., on Nov. 12 at the age of 71. He graduated from West Point in 1885 and spent two years at Detroit as assistant engineer on rivers and harbors and returned to West Point in 1889 and served for five years as instructor in civil and military engineering. After being promoted to captaincy in 1896 he was assigned to duty in the chief engineer's office in Washington. From 1900 to 1903 he was engineer in charge of buildings and grounds at the Military Academy at West Point.

After having spent several years as military observer in foreign countries, in 1906 he was given charge of fortifications, rivers and harbors at Norfolk, Va., and after becoming a lieutenant colonel was assigned to harbor work in Philadelphia and was shortly after given the command of the Engineering School at Washington. From 1915 to 1917 he served as military observer in Germany and on his return was made Chief of the War College and given the rank of brigadier general. He then entered active service in military operations

during the war. In 1925 he was made major general a few months before his retirement.

MICHAEL J. MORRIS, vice president of the contracting company of Atwell, Dustin & Morris, Inc., died at his home in Hollis, L. I., on Nov. 14 at the age of 70. Mr. Morris was born in England and came to this country at an early age and became engaged in construction work and railroad building. 1898 he went with the Degnon Contracting Co., Inc., New York City, as general superintendent on subway work in Manhattan. He remained with this company until 1923 and in 1925 formed the company of which he became vice president.

HARRY G. CLARK, executive vice president of the Rock Island R.R., died at his home in Glencoe, Ill., on Nov. 12, at the age of 60. Mr. Clark graduated with a civil engineering degree from the University of Kansas in 1898. He began his engineering work as a chainman with the Atchison, Topeka & Santa Fe Ry., and in 1900 joined the Rock Island system. He later became engineer of the Choctaw, Oklahoma & Gulf Division and the Panhandle & Arkansas Division. He then became train master of the Arkansas & Oklahoma division and later achieved the positions of assistant vice president and chief engineer of the entire system. He then became assistant to the president and in 1927 was made executive vice president.

CONSTRUCTION STATISTICS FOR THE WEEK

THE HEAVIEST state and municipal engineering construction awards in over two years push this week's construction volume total up to $51,646,000. Of this amount $6,334,000 is for private work and $45,312,000 for public. Of the public, $4,829,000 is federal and $45,312,000, state and municipal. Corresponding figures a year ago are: total, $37,407,000; private, $3,227,000; public, $34,180,000; federal, $17,189,000; state and municipal, $16,991,000.

Sewerage and highway awards are largely responsible for this increase. The classified totals are: streets and roads, $14,430,000; sewerage, $12,371,000; bridges, $2,801,000; earthwork, irrigation, waterways, $2,750,000; waterworks, $1,462,000; public buildings, $4,742,000; commercial buildings, $3,314,000; industrial buildings, $1,832,000.

The larger awards for the week include: memorial theater at Stanford University, Palo Alto, Calif., $585,000; U. S. Veterans Hospital, Waco, Tex., $1,128,000; relaying curbs, walks, gutters, fencing, grading 99 streets, relief labor, Yonkers, N. Y., $652,000; highway awards by Illinois, $545,000; Michigan, $474,000; Missouri, $637,000; New York, $1,376,000; Georgia, $857,000; Kentucky, $1,684,000; Louisiana, $568,000; North Carolina, $530,000; bridges by Montana, $503,000; and by New York, $856,000; sanitary sewers by WPA labor, St. Louis, Mo., $500,000; storm drains, WPA labor, Los Angeles, Calif., $4,256,000; sewers and storm drains, WPA labor, Los Angeles, $3,023,000; First unit of Hyperion, Sewage Disposal Plant, Los Angeles, Calif., $3,132,000; dredging in Caloosahatchee River, Florida, two contracts, $697,000; repairs, Mississippi River levees, Louisiana, $657,000; 587 mi. electric distribution system for Indiana Statewide Rural Electric Membership, c/o Indiana State Farm Bureau, $568,000; Newark, N. J., airport, including hangars, drainage system, and landscaping, $2,195,000; swimming pool, Broadhurst Ave., 146th to 147th Sts., New York, N. Y., $1,050,000; and 20,000 tons steel rails for L. and N. R.R., Louisville, Ky., $700,000.

New capital includes $7,698,000 in state and municipal bond sales and $57,691,000 added out of the WPA allotments released during the week.

CONTRACTS

(Thousands of Dollars)

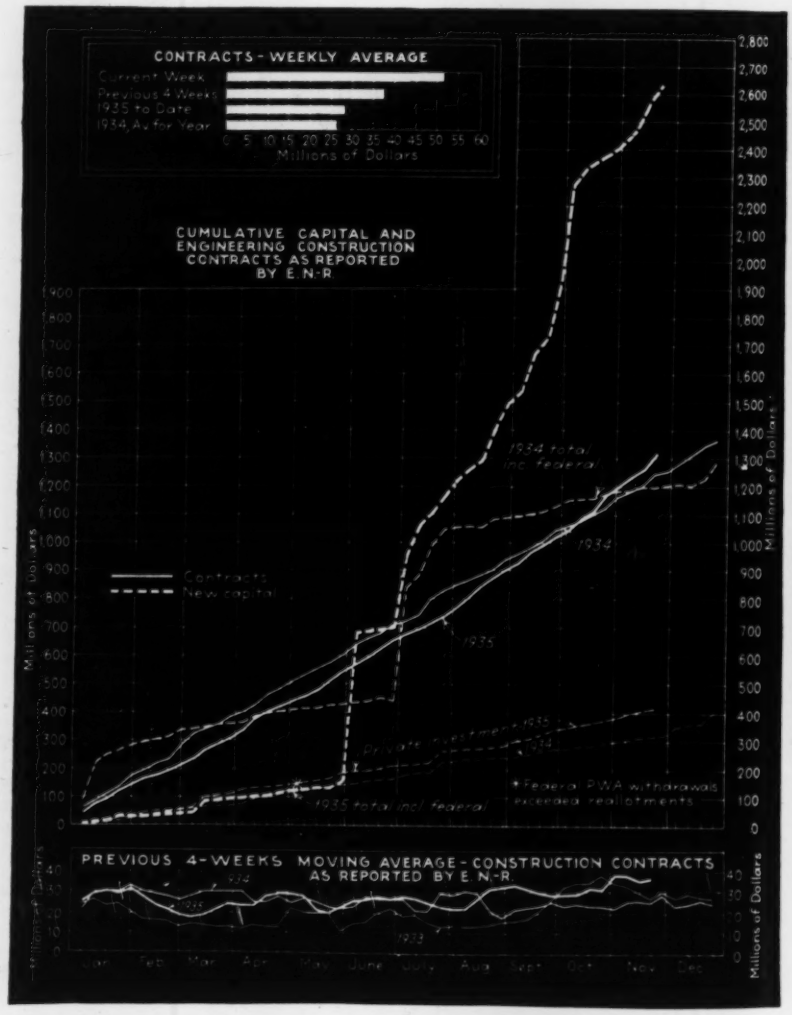
	Weekly Average Week Nov. 1934	Prev. 4 Week	Nov. 21 1935
Federal Government	$6,333	$5,921	$4,829
State and municipal	16,724	22,008	40,483
Total public	$23,057	$27,929	$45,312
Total private	3,826	9,264	6,334
Week's total	$26,883	$37,193	$51,646
Cumulative to date: 1934	$1,242,953,000	1935	$1,312,782,000

NEW PRODUCTIVE CAPITAL

(Thousands of Dollars)

1935	Week Nov. 21	Cumulative
State and municipal	$7,698	$383,170
PWA allotments, S&M	.....	786,169
RFC loans, S&M	.....	48,180
Corporate issues	.....	27,738
PWA private	.....	9,086
Local contrib. to WPA	.....	31,861
Total non-federal	$7,698	$1,286,204
PWA federal	57,691	1,346,627
Total new capital	$65,389	$2,632,831
Cumulative to date: 1934	$1,199,242,000	1935	$2,632,831,000

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for federal construction, deficiency act funds. An arbitrary percentage (25%) of the WPA allotments and local contributions to WPA work is included to allow for the capital additions through the Works Progress Administration division of the new program.

INDEX NUMBER

ENR Cost	1913 = 100	1926 = 100	ENR Volume	1913 = 100	1926 = 100
Nov., 1935	**194.90**	**93.69**	**Oct., 1935**	**162**	**71**
Oct., 1935	195.10	93.82	Sept., 1935	127	56
Nov., 1934	201.36	96.76	Oct., 1934	101	44
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	102	47
1932 (Av.)	156.97	75.45	1932 (Av.)	127	56